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Demonstration of Void Space Storage with Treatment and Flow Regulation

Karl R. Rohrer Associates, Inc, Akron, Ohio

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Environmental Protection Agency, Cincinnati, Ohio

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The prototype facility was constructed in Akron, Ohio with a combined sewer drainage area of 76.3 hectare (188.5 acres). The excess combined sewer flows were fed by gravity into the 3.8 x 10^6 liter (1 MG) void space retention tank. The tank is of an excavated hopper shape, lined with an impermeable membrane and filled with an inert media. Storage of the waste water is in the void space of the media. After the storm event, the stored stormwater was gravity fed into the interceptor sewer for subsequent treatment. The underground facility was a dual usage concept. In addition to collecting, chlorinating, and detaining potential combined sewer overflows, the facility's top surface could be made usable as a park or recreational grounds.

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DEMONSTRATION OF VOID SPACE STORAGE
WITH
TREATMENT AND FLOW REGULATION

by

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Project No. 11020 DXH

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report describes one method of prevention of combined sewer overflows by storing potential overflows in an underground void space retention tank. The collected wastewater is discharged, at a regulated rate, to the treatment plant during non-peak hours to prevent overloading of the treatment plant facilities.

Francis T. Mayo
Director
Municipal Environmental Research Laboratory

ABSTRACT

The purpose of this project was to demonstrate and evaluate the feasibility of an underground void space storage tank, in containing and regulating storm overflows from a combined sewer thus reducing the pollution loads discharged to the receiving water body. System design, construction, and two years operation were conducted under the study.

The prototype facility was constructed in Akron, Ohio with a combined sewer drainage area of 76.3 hectare (188.5 acres). The excess combined sewer flows were fed by gravity into the 3.8×10^6 liter (1 MG) void space retention tank. The tank is of an excavated hopper shape, lined with an impermeable membrane and filled with an inert media. Storage of the wastewater is in the void space of the media. After the storm event, the stored stormwater was gravity fed into the interceptor sewer for subsequent treatment. The underground facility was a dual usage concept. In addition to collecting, chlorinating, and detaining potential combined sewer overflows, the facility's top surface could be made usable as a park or recreational grounds.

This report was submitted in fulfillment of Project No. 11020 DXH between the United States Environmental Protection Agency (U.S.EPA) and the City of Akron, Ohio

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ABBREVIATIONS

BOD	biochemical oxygen demand
c	coefficient of runoff
cfs	cubic feet per second
cm	centimeter
COD	chemical oxygen demand
CY	cubic yard
DO	dissolved oxygen
DWF	dry weather flow
D_{10}	stone size of which 10% is finer
fps	feet per second
ft.	feet
gal.	gallons
GPM	gallons per minute
HP	horse power
hr	hour
i	rainfall intensity
in.	inch
k	permeability of stone
km	kilometer
l	liter
l/sec	liter per second
LF	lineal feet
LS	lump sum
m	meter
mg	milligram
MG	million gallons
MGD	million gallons per day
N.D.	not detectable
psi	pounds per square inch
P.V.C.	polyvinyl chloride
Q	flow rate
SS	suspended solids
SY	square yard
t	rainfall duration
U.S. EPA	United States Environmental Protection Agency

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SECTION I

INTRODUCTION

PURPOSE OF THE PROJECT

Combined sewer systems, which make provision to overflow excess amounts of stormwater during precipitation events, were adopted by many cities throughout the United States due to the economic savings incurred when compared to separate storm and sanitary sewers. Untreated combined sewer overflows have high concentrations of 5-day Biochemical Oxygen Demand (BOD) and suspended solids (SS). The average BOD concentrations in combined sewer overflows are approximately one-half that of raw sanitary sewage; the average SS concentrations are approximately twice that of the raw sanitary sewage.

Methods are being studied on the elimination of the pollution problem associated with combined sewer overflows. In addition to being a costly alternative, sewer separation does not necessarily alleviate the problem. Even separate storm wastewaters are significant sources of pollution, typically characterized with SS concentrations equal to or greater than those of untreated sanitary wastewater and BOD concentrations approximately equal to those of secondary effluent.

This project demonstrates and evaluates the feasibility of an underground void space temporary storage tank, as one solution to the pollution problem presented by combined or storm sewer overflows. By intercepting the combined sewer overflows, storing them in the void space retention tank, and regulating the discharge of the detained stormwater to the treatment facility during nonpeak hours, a significant degree of pollution reduction at the outfall can be attained. All stormwater which would exceed the storage capacity of the facility, would be chlorinated before being discharged into the receiving waterbody.

The underground storage facility is a dual usage concept. In addition to collecting, storing, and regulating potential combined sewer overflows, the facility's top surface could be made usable as a park or recreation grounds. This dual usage concept establishes an aesthetically pleasing and socially acceptable storage facility.

SCOPE OF PROJECT

The scope of the project can be divided into three distinct areas. The first area of work consisted of:

1. site investigation and acquisition of property
2. topographic and hydrographic surveys
3. hydraulic studies
4. structural and instrumentation design
5. specifications and construction drawings.

The second area of work consisted of:

1. construction of the prototype facility.

The third area of work consisted of:

1. operation of the prototype facility for two years
2. evaluation of the operational period
3. continued operation of the facility by the city.

SECTION II

CONCLUSIONS

Much emphasis has been placed on the pollution load to receiving water bodies caused by combined sewer overflows, and many methods have been studied on controlling and preventing these overflows. Determination of the most economical approach must consider the available methods: sewer separation, storage, treatment, and storage with treatment. The underground void space storage concept can be one economically feasible and aesthetically appealing solution to the abatement of combined sewer overflows.

1. The total capital cost of the 3.8 million liter (1 MG) demonstration project was $750,000. Of this amount, $471,000 was used in the construction and instrumentation of the prototype facility. It is anticipated that future facilities, with limited instrumentation for storm evaluation, could be constructed for $0.079 to $0.092 per liter ($0.30 to $0.35 per gal.).

2. The underground void space storage concept allows for dual land usage. This versatility permits adaptation of the surface for recreational ground, parking, open-air storage, or other facilities which make it compatible with its surrounding areas.

3. No detectable offensive odors or flammable gases were generated in the storage facility during the two year operational period. A combination of factors was responsible for this: the storm flow from this steeply sloped combined sewer system consisted mainly of suspended inorganic solids rather than putrescible organic materials, chlorination of the flow entering the media prevented the wastewater from going septic, and the detention times of the stored overflows were relatively short. Slime and algal growth within the media were not detected, and there were no rodent or insect infestations.

4. The first flush concept, produced by settleable material being deposited within the combined sewers during dry-weather flow, was not evident for the Tallmadge Memorial Parkway drainage area. The high dry-weather flow velocities associated with the steep sloped (9%) drainage area,

prevented material from settling in the sewers. At the onset of a storm, however, there are high initial concentrations of suspended solids due to the sediment which is carried into the inlet basins as runoff from the drainage area.

5. Based on the calculated volume reduction due to contained solids, and an average of 10 complete fillings per year, over 65 percent of the void space volume would remain after 25 years of use. These calculations assume that all of the suspended solids which enter the media would be contained.

6. The prototype facility was not designed to operate automatically, hands-off, thus many storms had ceased before an operator arrived at the facility. Due to the drainage area's short initial time of concentration (20 min.), the variable nature of storms, and the difficulties experienced with the storm notification system, many precipitation events were not stored in the facility and operational data was not collected.

Although the prototype facility experienced some operational difficulties and several design changes are recommended, the void space storage concept could be a practical and economical solution to the pollution problems associated with combined sewer overflows.

SECTION III

RECOMMENDATIONS

The primary purpose of the two-year study and evaluation period of the Tallmadge Memorial Parkway Storm Water Retention Tank #1, was to determine the feasibility of the underground void space storage concept (Geo-Cel) in the reduction of pollutant loads from combined sewer overflows on receiving water bodies. Based on the results obtained from the prototype facility, several design alterations are presented which would increase the potential and cost-effectiveness of the void space storage concept.

1. Due to the insufficient data obtained from the influent sampler, the effectiveness of the settling tube modules could not be completely evaluated, however, because of the high and variable flow rates of stormwater entering the system (greater than those recommended by the manufacturer), they did not prevent the carryover of suspended solids onto the exposed media surface. For systems involving flow rates greater than the tube settler's design rate, the modules should not be incorporated into the facility design.

2. If the settling tube modules would be eliminated from the system design, the clarifier, which comprised nearly one-fourth of the total construction costs, could be drastically reduced in size. This, however, would require an alternative means of solids removal or a redesigned storage cell feed system to prevent the detrimental effects of solids reducing the infiltration rate into the exposed media surface.

   Where natural media (stone) is used, the storage cell should have a bottom or well feed system which would be partially self-flushing after each storage event. If an economical artificial inert media could be obtained, which would have greater void space and less surface contact points, any feed system would be adequate since the suspended solids would not be captured on the exposed media surface.

3. The present 1.22 meter (48 in.) riser overflow system and the 1.52 meter (60 in.) emergency pressure relief gate valve could be redesigned to allow a more economic and

automatic operation. If and when the suspended solids would reduce the infiltration rate into the media, the overflow system should automatically provide pressure alleviation.

4. The combined sewer influent flow should be monitored with a direct readout flow meter. In addition to giving instantaneous influent rates, the flow meter signal (electrical or pressure) could be used to provide a precise automatic chlorination input system.

5. Due to the turbidity and color of the combined sewer flows, a non-colorimetric chlorine analyzer is suggested if an analyzer is to be used in the facility.

6. Future study of detention time of combined sewer overflows should be conducted to determine the deleterious effects of odor and combustible gas as a function of time.

7. Both liner materials tested proved effective for the duration of the prototype facility's evaluation period. Continued use of the facility by the City will provide additional information on the life expectancy of the two liners. Where good site conditions prevail, future installations could use the more economical liner.

Future studies should be performed which would focus on analyzing a series of void space retention tanks at combined sewer outfalls for their joint effects on the water quality of the receiving watercourse. The sampling program should include both a pre- and post-installation evaluation of the receiving river or stream water quality, in addition to the monitoring and sampling of the stored combined sewer overflows.

SECTION IV

TEMPORARY STORAGE FOR POLLUTION REDUCTION

Within the past decade increasing awareness and concern of the pollution factors in the environment have focused attention on the pollution load contributed to the nation's water resources by combined sewer systems. Many cities adopted the combined manner of sewer construction due to economic savings when compared to separate storm and sanitary sewers.

Methods are constantly being studied on how to eliminate the pollution problem presented by combined sewer overflows. Separation of sewers is not only economically unfeasible but also does not allow treatment of the storm runoff which is recognized as a major contributor of pollutants to the receiving water body. The U.S. EPA report 11022 ECV 09/71 entitled "Underwater Storage of Combined Sewer Overflows" states:

> "Separation of sewers may not be the economical answer for elimination of combined sewer pollution load. From studies conducted during the past decade, evidence is that the total pollution load from storm overflow from combined sewer systems is not from sanitary flow only. About 80% of the pollution load is due to the sanitary wastes. The other 20% of the pollution load is entirely due to storm runoff."

Both the quantity and quality of stormwater runoff from an urban area are highly variable and transient in nature, being dependent upon meteorological and climatological factors, hydraulic characteristics of the surface and underground conduits, and on the nature of the antecedant period. The pollutants contained in urban stormwater runoff and drainage include street litter, leaves, grass clippings, dust and debris of airborne fallout, eroded soil, animal excreta, fertilizer, insecticides, road salt and other chemicals.

Combination sewer systems have three major drawbacks:

1. The increased flow during storms may overload the hydraulic capacity of the sewage treatment plant.

2. Low velocities of dry-weather flow promote high loadings of solids and BOD to settle in the sewers, promoting a septic condition.

3. The "first flush" of stormwater including these solids may be discharged directly into the receiving body of water through the regulating device, or may be flushed to the treatment plant where it upsets the biological balance.

Although combined sewers may provide adequate capacity to carry peak storm flows, sewage treatment facilities do not have the same hydraulic capacity. During periods of precipitation, the sewage treatment facility may be unable to cope with the combined flows; the excess is diverted from the treatment plant and discharged directly to a receiving water or water course. These combined sewer overflows are of particular concern because they result in the bypass to the environment of untreated sewage and present a potential public health hazard.

Advocates of partial storage systems point out that the major pollutant load encountered in storm overflow from combined sewers occurs during the first flushing action of a storm. If storage can be provided to accommodate this first flush of water, a high degree of pollution control will be accomplished at much less capital investment than with sewer separation or total storage systems.

Due to the nature of combined sewer overflow, that is, a short duration, high flow volume discharge, it appears that, in many cases, a storage cell could be incorporated into the overall system or scheme to control pollution from combined sewer overflows.

For those treatment facilities containing high-rate processes such as mechanical screening and dissolved air flotation, the least costly facility always resulted when the treatment process was combined with a holding pond whose capacity was matched with that of the process to provide a facility of optimum size. The holding pond provides mainly peak flow attenuation, but also some pollutant removal capability as well. Reduction of instantaneous peak flows results in both smaller downstream treatment facilities with improved utilization factors, and lesser instantaneous pollutant loadings and effects on receiving waters. For example, if an appropriately sized storage facility was placed strategically at the point of overflow, ahead of a municipal water pollution control plant, it may be possible to accommodate the combined sewer overflow discharge over an extended period of time. The storage facility would return the storm drainage, which would have otherwise overflowed, to the municipal treatment plant some time after the storm overflow had ceased. The storage treatment facility would be sized to treat a certain peak volume of combined sewer overflow, which can be determined from an analysis of the combined sewer overflow hydrographs for a defined drainage area. All overflows in volume equal to or less than the design capacity of the storage-treatment facility would be handled without any untreated pollutant discharge to the receiving waterbody. Determination of the required capacities of a storage facility to contain stormwater runoff and combined sewer overflows associated with specific storm recurrence intervals must be based upon the treatment facility's hydraulic capabilities, drainage area topography, rainfall intensity probabilities, and future runoff coefficient projections.

The appropriate location for future overflow storage-treatment facilities would be in the vicinity of the present overflows or diversion structures and, in certain locations, where large trunk sewers meet the interceptor, even though an overflow structure does not now exist. The facility is used as an interim storage device to stop overflows, hence possibly alleviating the need for an expensive program to meet current effluent standards.

This project demonstrates and evaluates one method for the reduction of combined sewer pollution. Excess stormwater is chlorinated, clarified and flows into a large, underground, lined tank filled with "media" (in this case gravel) which holds up the roof. The water is stored in the void space of the media until the storm subsides. This concept of void space storage in an underground, lined tank filled with inert media is called a Geo-Cel.

Pollution reduction is accomplished by containment and delay with chlorination in the Geo-Cel. Combined sewage, which would normally flow into the receiving waterbody, is contained within the storage facility until the sewage treatment plant has the capacity to process the Geo-Cel effluent. The facility effluent can be delayed and regulated by manipulation of the effluent slide gates.

The storage holding basin concept will work on both combined and storm sewers should there be sewer separation.

Void space storage, where applicable, offers low land costs through dual land usage, an aesthetically pleasing and compatible installation, and an economical method of elimination of combined sewer pollution.

By dual use of the top surface, it is possible to incorporate aesthetically appealing landscaping, and possibly recreational facilities, such as playgrounds or tennis courts. The added cost of landscaping or recreational facilities would not be high from the standpoint of the project, but would substantially increase the social acceptance and social value of the project. Parking space is also another potential usage.

Geo-Cel temporary holding tanks should be considered as one of the alternative approaches to regaining system storage in the future, when population and industrial growth have encroached upon the current excess capacity (if any) built into the collection system.

SECTION V

RAINFALL AND SITE CONDITIONS

RAINFALL AND PRECIPITATION

Prior to construction of the test facility, available recorded rain data for the area was investigated. Four different sources were available from the National Weather Records Center in Asheville, North Carolina. Table 1 lists the four sources, their respective length of record, and their location with respect to the site.

TABLE 1. RAINFALL STATIONS

Source	Data From-	To	Location
1. Akron Water Pollution Control Station	4-13-1957	Present	4.55 km (4.16 miles) N.N.W. of the site Elev = 232 meters (760 feet)
2. Cuyahoga Falls	10-2-1939	12/1941	4.55 km (4.16 miles) N.N.W. of the site
3. Akron Municipal Airport	1946	1952	4.62 km (4.22 miles) S.E. of site
4. Akron-Canton Airport	1948	Present	15.26 km (4.22 miles) S.E. of site Elev = 369 meters (1209 feet)

The existing data was used in the analysis of storm events for the Akron, Ohio area. The City of Akron, Bureau of Engineering has previously analyzed recorded rainfall data and formulated intensity - duration - frequency relationships for the Akron area. The intensity formula is of the form:

$$I = cT^m / 2.54\ (t+d)^n$$

Where: I is the intensity in centimeters per hour

t is the duration in minutes

T is the frequency of occurrence in years

and: c,d,m, and n are regional constants

These curves are represented in Figure 1 for recurrence intervals of 1,3,5,10,15,25,50 and 100 years and durations from 5 to 180 minutes. These relationships are less accurate for durations less than 5 minutes or greater than 180 minutes.

Also useful in the analysis of precipitation are two reports. The first report, by Marvin E. Miller, State Climatologist, and C.R. Weaver, was Monthly and Annual Probabilities for Selected Locations in Ohio published in March, 1969, by the Ohio Agricultural Research and Development Center in Wooster, Ohio. Curves were prepared for finding the probability of receiving a depth of precipitation for a given return period for any given month and annually for the Akron area. These curves are represented in Figures 2 and 3.

The second report contains the results of a study conducted by Marvin E. Miller. Curves were prepared for finding the maximum daily precipitation of a given return period for any given month and annually for the Akron area. These curves are represented in Figures 4 and 5.

The yearly maximum 24-hour precipitation for the 1887 to 1974 period for the Akron area are listed in Table 2.

DRAINAGE AREA AND SITE CONDITIONS

Within the lower, eastern, portion of the Tallmadge Memorial Parkway drainage area (Figure 6), bounded by the B & O Railroad on the west, Tallmadge Memorial Parkway on the south, and the Little Cuyahoga River on the east is a sector of land owned by the City of Akron. This land, which was in part used for sanitary landfill started in 1963, was allocated by the City of Akron for the construction and evaluation of the project: "Demonstration of Void Space Storage with Treatment and Flow Regulation". A topographical drawing of the drainage area is included as Figure 7.

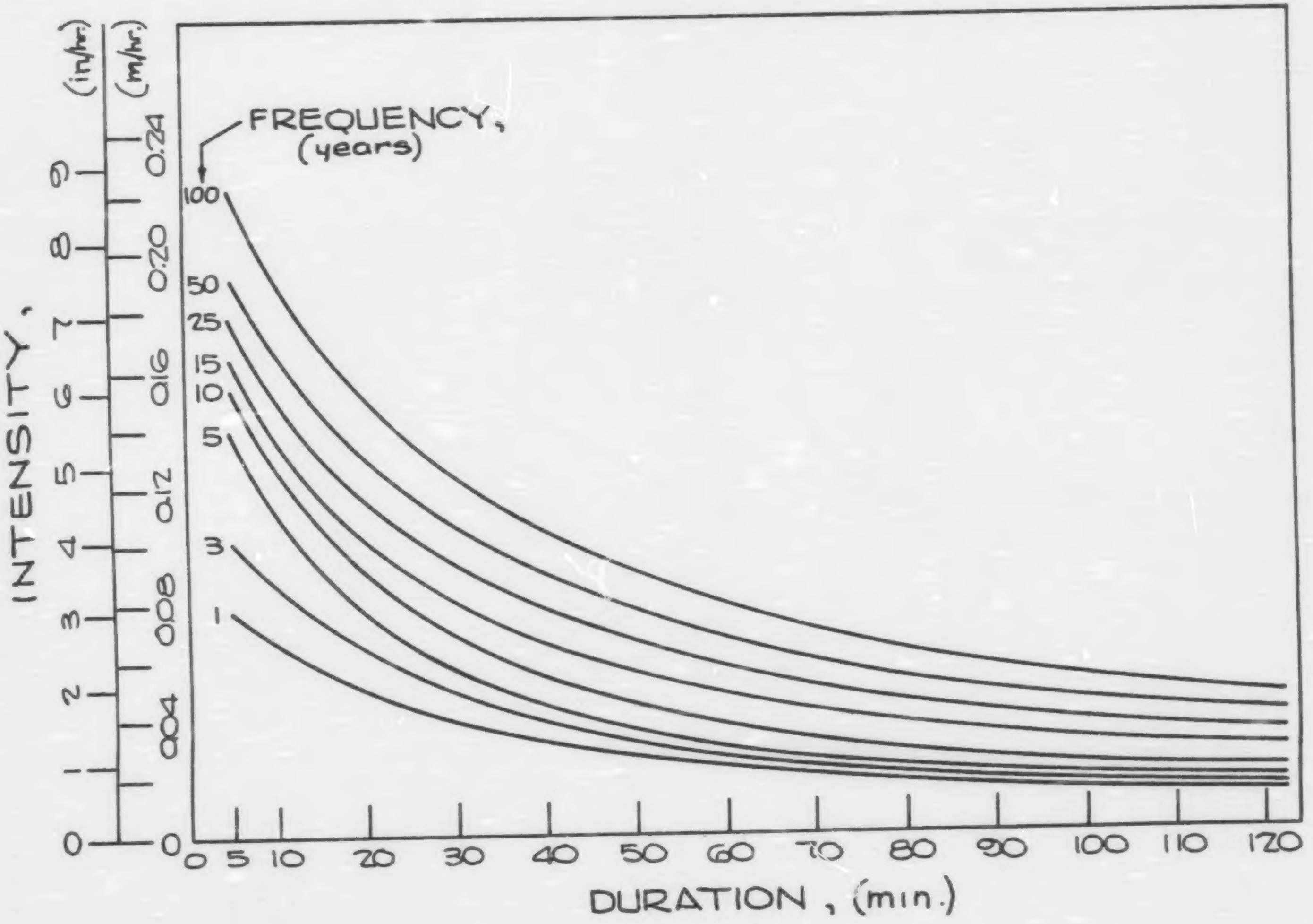


FIGURE 1. INTENSITY-DURATION-FREQUENCY CURVES FOR AKRON

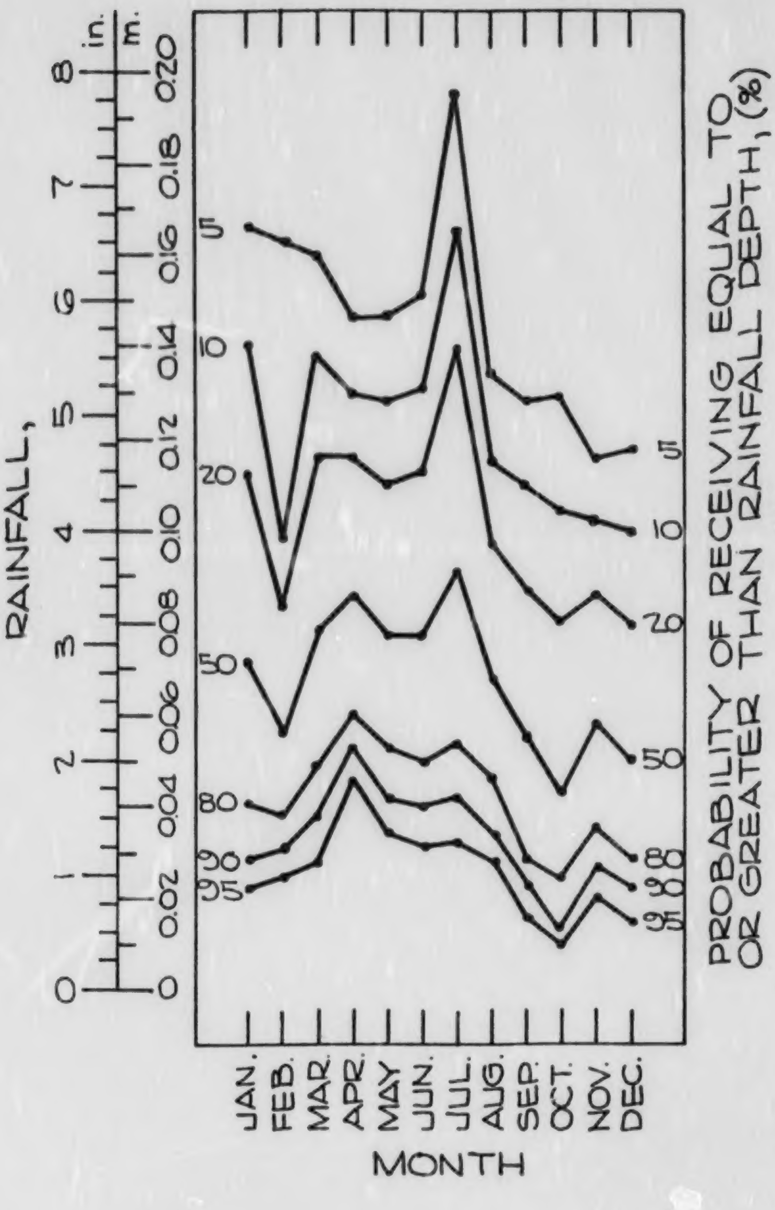


FIGURE 2. MONTHLY RAINFALL PROBABILITY CURVES

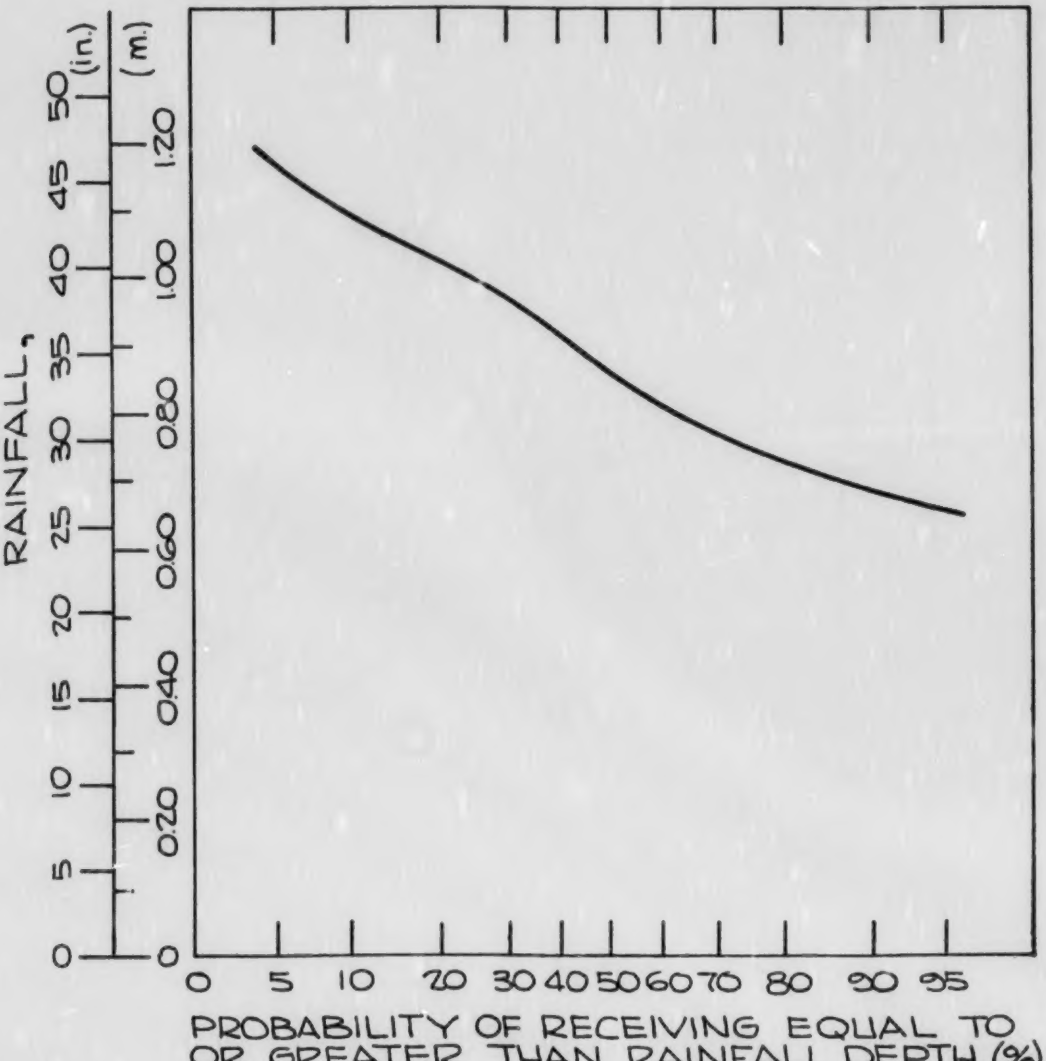


FIGURE 3. YEARLY RAINFALL PROBABILITY CURVE

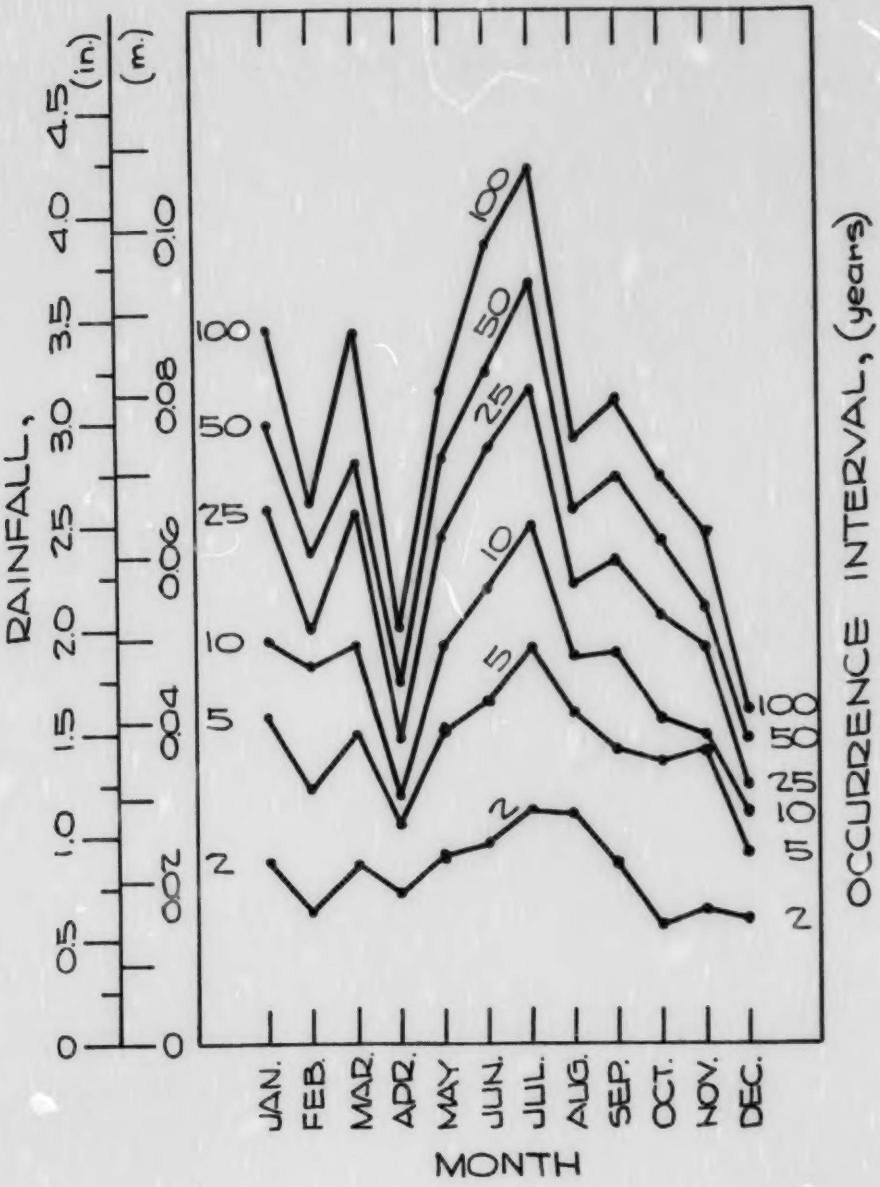


FIGURE 4. MONTHLY MAXIMUM DAILY PRECIPITATION

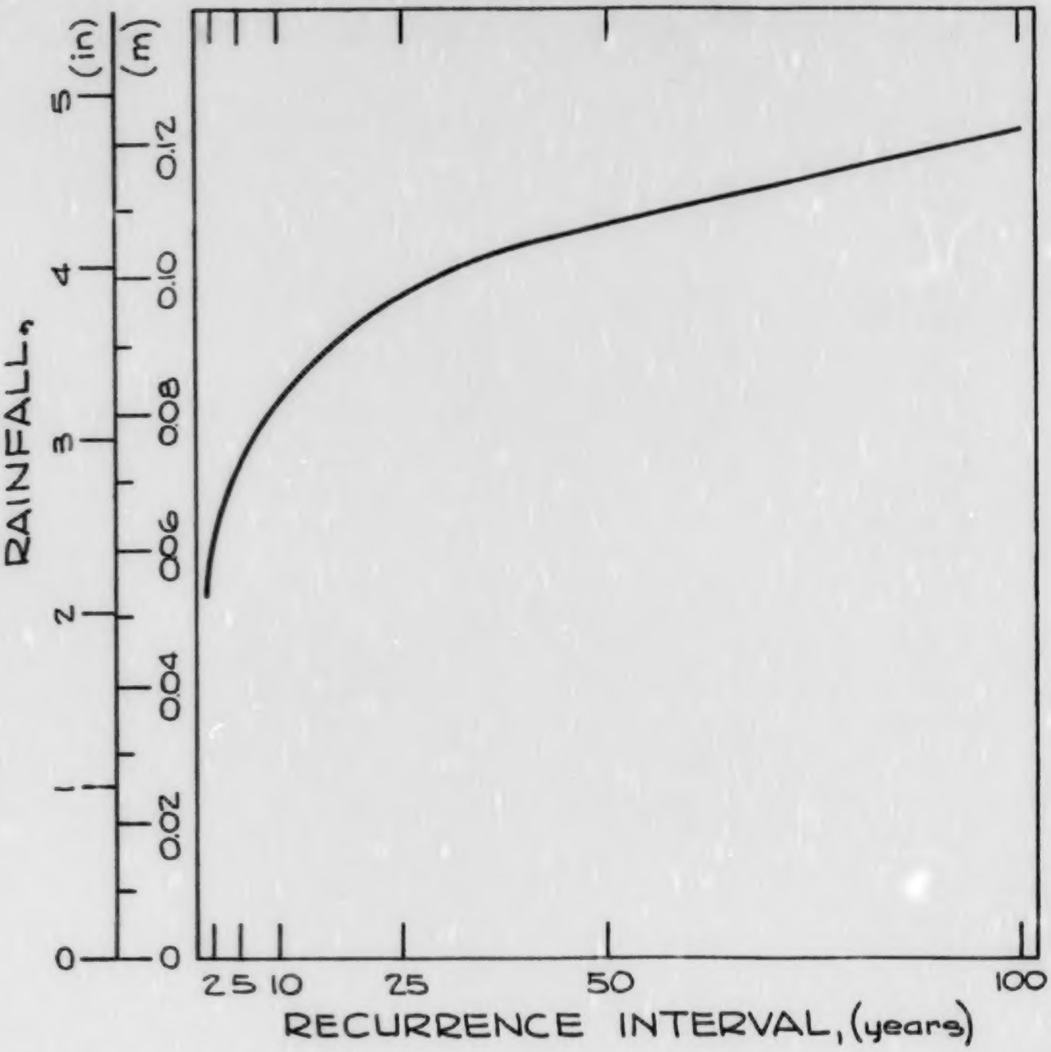


FIGURE 5. YEARLY MAXIMUM DAILY PRECIPITATION

TABLE 2. MAXIMUM RECORDED RAINFALL - AKRON AREA

Year	Month	Precipitation 24 Hour Period (cm)
1887	Aug. 6	4.24
1888	Aug. 4	4.24
1889	June 28	4.98
1890	Spet. 11	7.75
1891	June 18	2.95
1892	June 19	4.14
1893	Oct. 13	4.64
1894	Sept. 8	4.17
1895	Aug. 6	4.50
1896	June 7	9.14
1897	July 5	5.21
1898	Nov. 5	4.62
1899	May 27	6.07
1900	July 25	5.84
1901	Aug. 19	4.78
1902	July 18	5.28
1903	Aug. 27	7.92
1904	April 1	4.87
1905	May 25	4.17
1906	July 5	3.07
1907	Jan. 4	3.73
1908	May 20	4.22
1909	Aug. 15	4.32
1910	Oct. 6	6.10
1911	Sept. 15	8.18
1912	July 14	7.85
1913	March 25	12.07
1914	March 24	4.70
1915	June 15	3.66
1916	May 29	13.03
1917	June 1	2.74
1918	May 13	3.94
1919	June 26	4.17
1920	June 17	6.73
1921	March 28	5.16
1922	May 19	3.73
1923	May 12	4.60
1924	June 29	10.92
1925	July 10	4.37
1926	Sept. 2	11.81
1927	Dec. 1	5.26
1928	July 20	7.16
1929	April 5	6.48
1930	Jan. 8	2.74
1931	Sept. 26	2.95
1932	Aug. 18	4.57
1933	March 13	7.11
1934	Sept. 15	5.56
1935	Aug. 2	4.83
1936	July 24	6.10
1937	January	4.75
1938	Sept. 12	11.05
1939	Oct. 27	4.98
1940	Aug. 27	4.88
1941	July 7	7.92
1942	May 15	3.91
1943	July 7	15.14
1944	Aug. 23	4.09
1945	May 17	7.24
1946	July 21	3.43
1947	Aug. 25	5.44
1948	Sept. 19	6.12
1949	July 9-10	4.83
1950	July 9-10	5.64
1951	Sept. 13	3.86
1952	Sept. 18-19	5.56
1953	June 16-17	6.32
1954	March 1	2.44
1955	Nov. 15	5.08
1956	May 26-27	6.86
1957	Feb. 24-25	5.70
1958	July 15	4.27
1959	July 18	4.52
1960	June 12-13	5.79
1961	July 19	8.15
1962	July 3	3.91
1963	Aug. 6	3.48
1964	July 21	7.75
1965	Aug. 25	3.81
1966	Nov. 2	4.37
1967	Sept. 27	3.89
1968	July 17	3.48
1969	July 5	6.10
1970	June 14-15	9.17
1971	Aug. 26-27	7.29
1972	July 15-16	8.33
1973	July 20-21	4.17
1974	Aug. 3-4	5.84

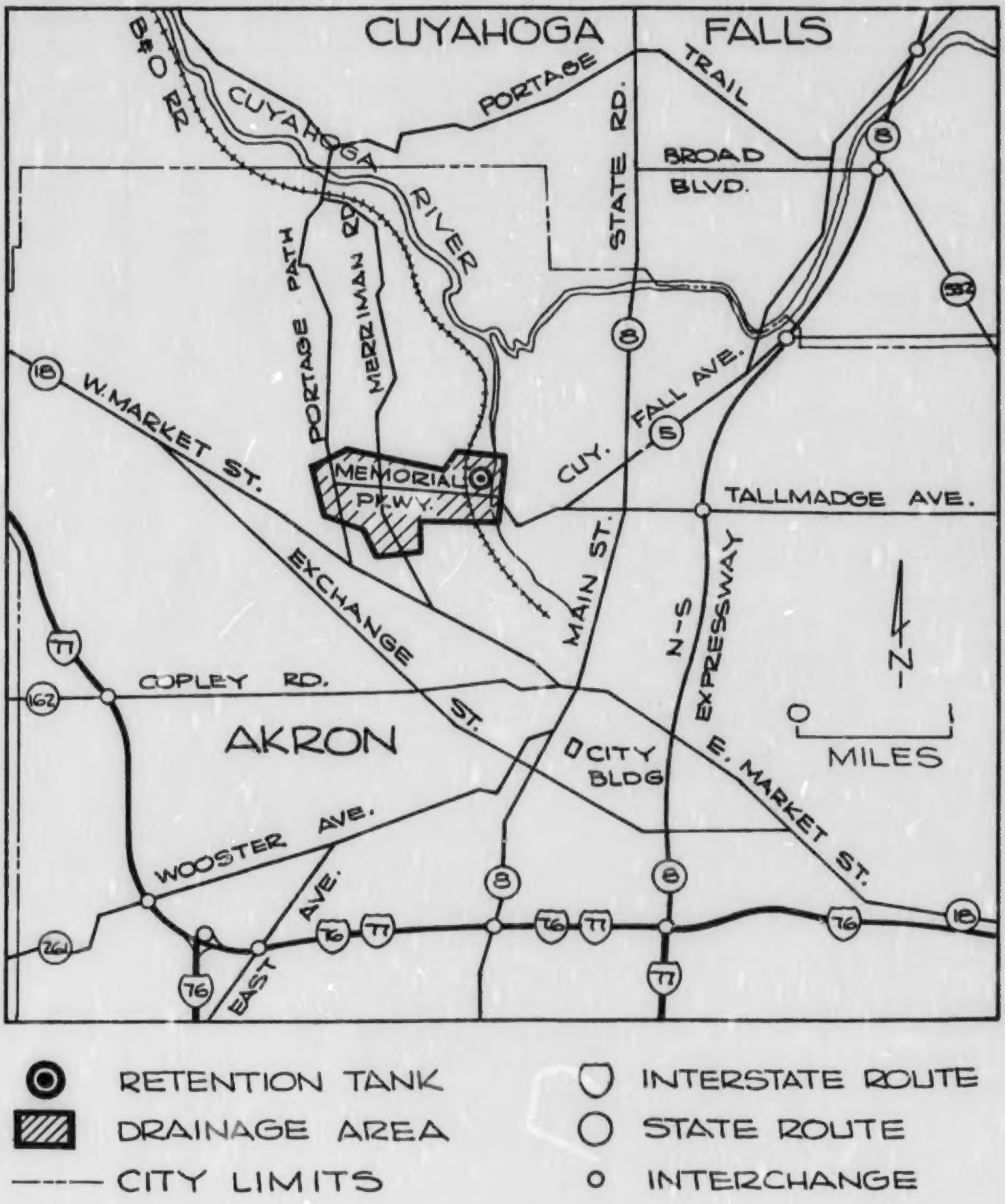


FIGURE 6. DRAINAGE AREA LOCATION

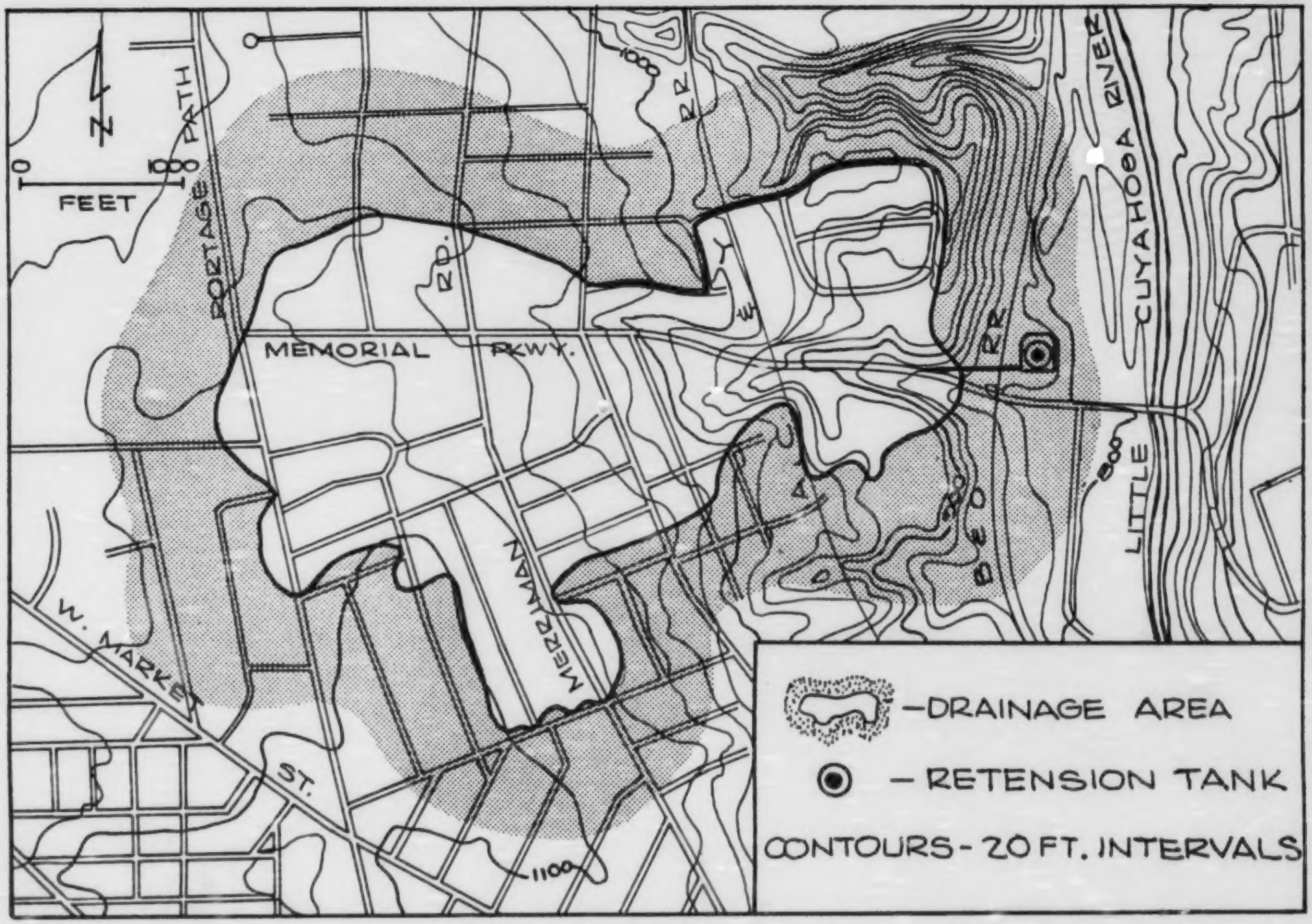


FIGURE 7. TOPOGRAPHIC DRAWING OF DRAINAGE AREA

The drainage area, which is generally an older but above average income neighborhood, is characterized by large lawn areas and steep slopes. Over 91 meters (300 ft.) of elevation difference are encountered between the area's highest point and the headwall at the Little Cuyahoga River. There are several apartment buildings and one primary school within the area. No commercial establishments, not even gasoline stations, are contributory to the drainage area.

The landfill, which was started in 1963 on the area adjacent to the retention tank, is approximately 2 km (1-1/4 mi.) long and an average of 0.4 km (1/4 mi.) wide. The depth of fill adjacent to the tank was 1.2 to 1.5 meters (4 to 5 ft.) and increased in depth to approximately 3 meters (10 ft.) near the Little Cuyahoga River. The area in which the retention tank is located was filled with a clean fill since some housing previously existed nearby. The intention was to keep the organic material further away from the housed area to reduce complaints of odors and pests. These houses have since been removed. In the area near the retention tank, all debris was covered with a minimum of 0.91 meters (3 ft.) of cover.

One of the reasons the City of Akron selected this area as a land fill was the fact that a blue clay stratum underlies the entire landfill, thereby reducing the leaching and pollution of ground water. The generalized stratigraphic sequence of rock in the area of the tank shows a Pleistocene series of till with a heterogeneous mixture of clay, sand and gravel with a predominance of clay. Previous to construction, it was anticipated that the ground water table at the tank was 6.1 meters (20 ft.) below the ground surface. The last fill dumped on the area adjacent to the retention tank was land filled in 1964.

Runoff coefficients for the 76.28 hectare (188.5 acre) residential drainage area vary from approximately 0.40 to 0.70 with a weighted average estimated at 0.60. This weighted coefficient was used to predict anticipated flow rates into the system for any intensity rainfall.

Based on the intensity-duration-frequency relationships for Akron, and the Rational Method of drainage computation, Figure 8 represents the intensity-duration-flow rate-and volume relationships for the Tallmadge Memorial Parkway drainage area. The weighted coefficient of discharge, c, for the drainage area of 76.28 hectares (188.5 acres) was assumed to be 0.6.

The Tallmadge Memorial Parkway area is one of 35 combined sewer drainage areas in Akron, Ohio. Most of the present combined sewer collection system was built during 1929 and 1930, with additional areas being added as necessary. The existing Tallmadge Memorial Parkway drainage area system consists of 0.20

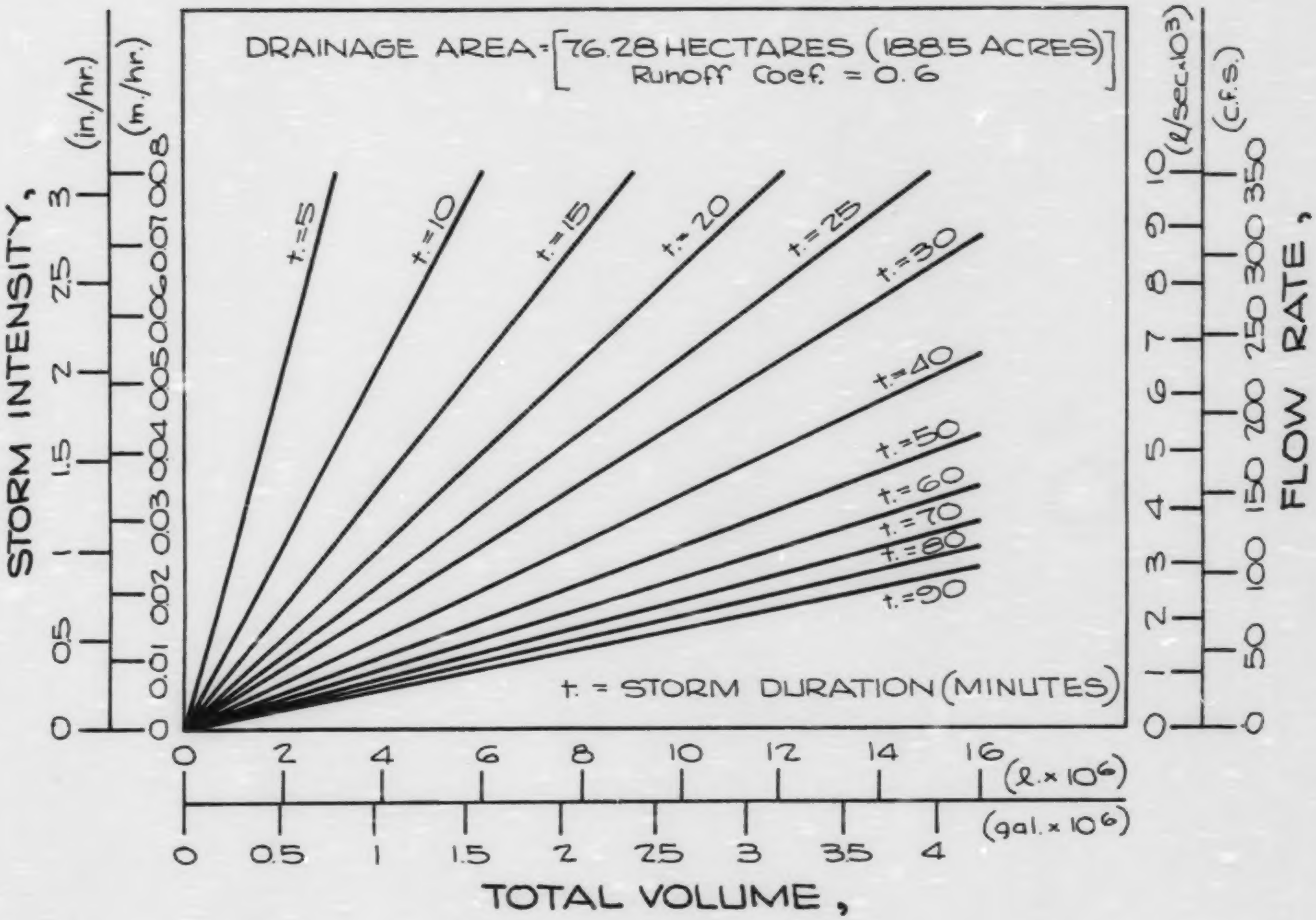


FIGURE 8. INTENSITY-DURATION-FLOW RATE-VOLUME RELATIONSHIPS

meter (8 in.) to 1.37 meter (54 in.) diameter sewers and a 1.37 meter x 1.37 meter (54 in. x 54 in.) rectangular storm overflow outlet.

The major trunk sewer through the drainage area varies from 0.38 meter (15 in.) to 1.37 meter (54 in.) diameter culverts. Vitrified clay tile is used for the small pipe diameters with brick and reinforced concrete being used for the larger sizes.

At the time of construction, the maximum dry-weather flow from the Tallmadge Memorial Parkway drainage area was estimated at 1.987 million liters per day (0.525 MGD) or 22.94 liters per second (0.81 cfs). The maximum flow from the drainage area to the Little Cuyahoga Interceptor was limited by a 0.305 meter (12 in.) pipe from the trunk sewer drop inlet to the interceptor. The 0.305 meter (12 in.) pipe had a capacity of 278 liters per second (9.8 cfs). All flow in excess of 278 liters per second (9.8 cfs) was discharged directly into the Little Cuyahoga River. Overflow into the river, therefore, could have been expected for any storm with rainfall intensities greater than 0.23 centimeters (0.09 in.) per hour.

SECTION VI

SYSTEM DESIGN

The "Demonstration of Void Space Storage with Treatment and Flow Regulation" consisted of construction, operation, and evaluation of an underground storage and treatment facility for combined sanitary and storm flows. The primary project facilities included a diversion manhole, a clarifier, and a 3.8 million liter (1 MG) temporary storage space. The wastewater was intercepted from the existing combined sewer system at the diversion manhole and was directed to the clarifier chamber of the retention cell by the 1.22 meter (48 in.) influent pipe. (Figures 9, 10, 13 and A-1 visually aid in the explanation of the system.) The clarifier was a large volume tank 379,000 liters (100,000 gal.) designed to reduce the high entrance pipe velocities and aid in the settling out of suspended sediment. Within the clarifier were tube settlers which increased the settling rate of suspended matter. The wastewater entered into the void space retention area by passing over the clarifier weir (above the tube settlers) and infiltrating into the exposed media surface (Figure A-1). The temporary storage tank included novel concepts in design and operation. It was a 60.96 meter x 60.96 meter x 3.05 meter deep (200 ft. x 200 ft. x 10 ft. deep) excavated hopper, lined with an impermeable membrane, filled with an inert media, covered with another impermeable membrane and soil, and the surface media made usable. The Tallmadge Retention Tank #1 used natural, non-fractured rock as the media with a void space of approximately 40 percent of the total volume.

With the occurrence of a storm, the flow rate into the facility increased in proportion to the rainfall intensity. If the flow rate of stormwater entering the clarifier was greater than the effluent rate, storage of excess stormwater automatically commenced. So long as the influent rate exceeded the effluent rate, regulated by the 0.46 meter (18 in.) and 1.52 meter (60 in.) slide gates, the volume stored within the clarifier increased until the water level reached the level of the weir. All additional flow caused overflow of the weir and sank into the 60.96 meter x 2.59 meter (200 ft. x 8.5 ft.) exposed media surface. The maximum storage space within the media was approximately 3.8 million liters (1 MG). If the volume of flow which passed over the clarifier weir exceeded 3.8 million liters, the chlorinated stormwater overflowed into the 1.22 meter (48 in.)

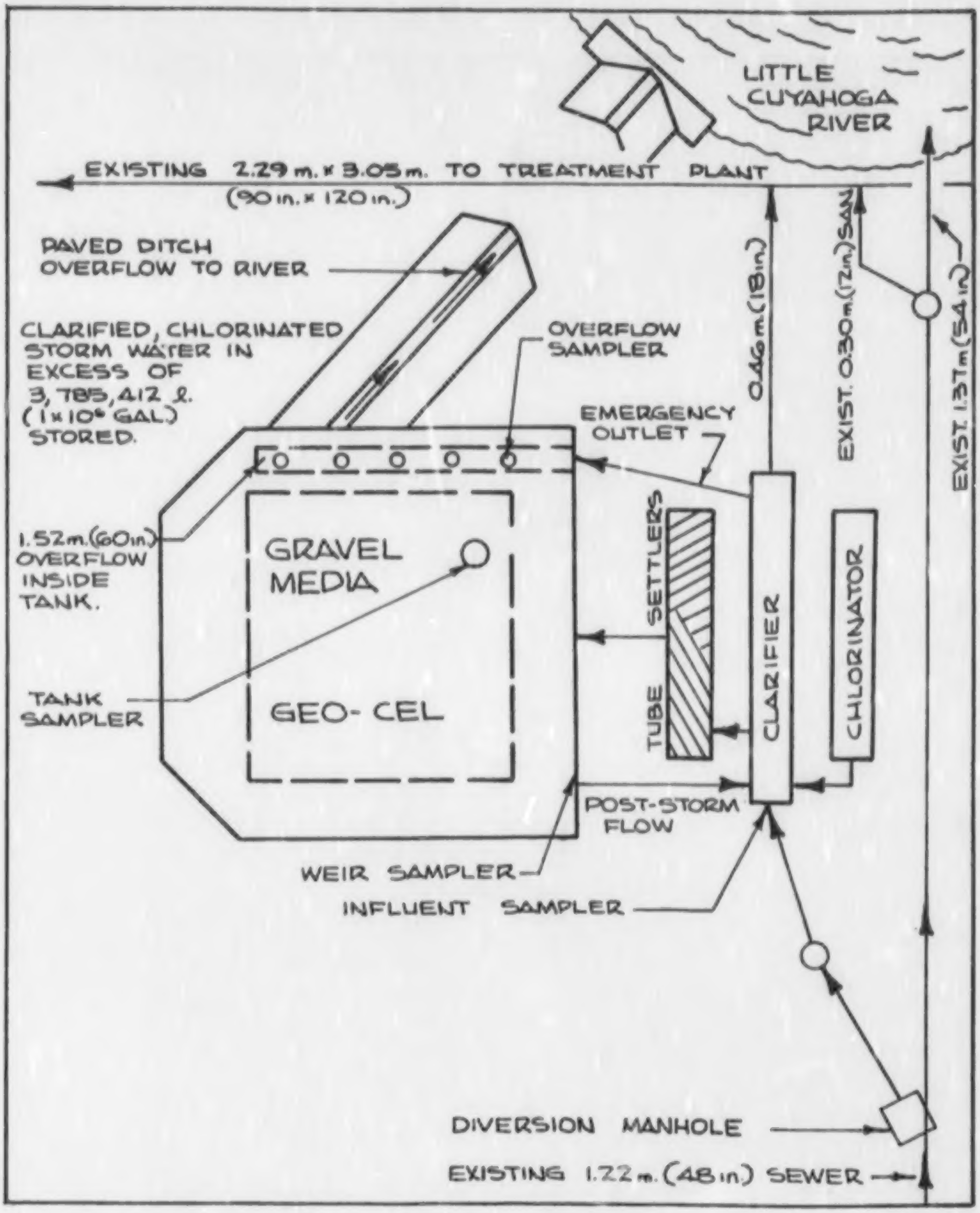

LITTLE CUYAHOGA RIVER
EXISTING 2.29 m. × 3.05 m. TO TREATMENT PLANT
(90 in. × 120 in.)
PAVED DITCH OVERFLOW TO RIVER
CLARIFIED, CHLORINATED STORM WATER IN EXCESS OF 3,785,412 ℓ. (1×10^6 GAL.) STORED.
OVERFLOW SAMPLER
EMERGENCY OUTLET
0.46 m.(18 in.)
EXIST. 0.30 m.(12 in.) SAN.
EXIST. 1.37 m.(54 in.)
1.52 m.(60 in.) OVERFLOW INSIDE TANK.
GRAVEL MEDIA
GEO-CEL
TANK SAMPLER
TUBE SETTLERS
CLARIFIER
CHLORINATOR
POST-STORM FLOW
WEIR SAMPLER
INFLUENT SAMPLER
DIVERSION MANHOLE
EXISTING 1.22 m.(48 in) SEWER

FIGURE 9. BLOCK DIAGRAM OF SYSTEM COMPONENTS

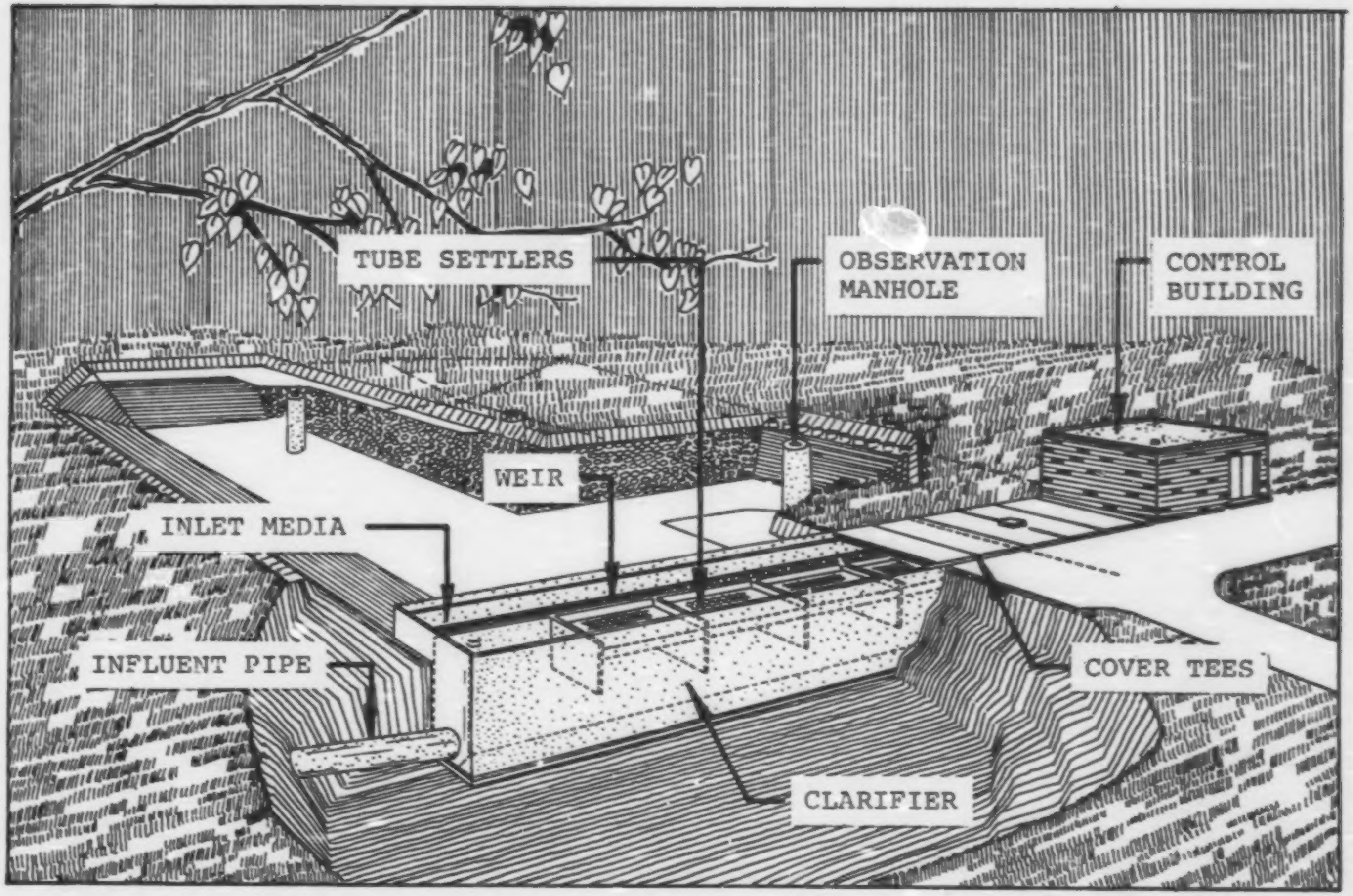


FIGURE 10. AERIAL CUT-AWAY PERSPECTIVE OF SYSTEM

risers and was transported to the Little Cuyahoga River. (The chlorinator was triggered when the water surface elevation reached that of the clarifier weir.) After the storm, the flow entering the clarifier gradually subsided to the dry-weather flow rate. That stormwater which was contained within the facility was gravity discharged, at a regulated rate, to the Akron Treatment Plant through the 0.46 meter (18 in.) effluent gate.

A more detailed analysis of each of the system components is presented in the following section, "System Components".

SECTION VII

SYSTEM COMPONENTS

Studies have been conducted on the various individual components of the facility to evaluate their operational feasibility and design. The basic project facilities included a diversion manhole, a clarifier, and a 3.8 million liter (1 MG) temporary storage space.

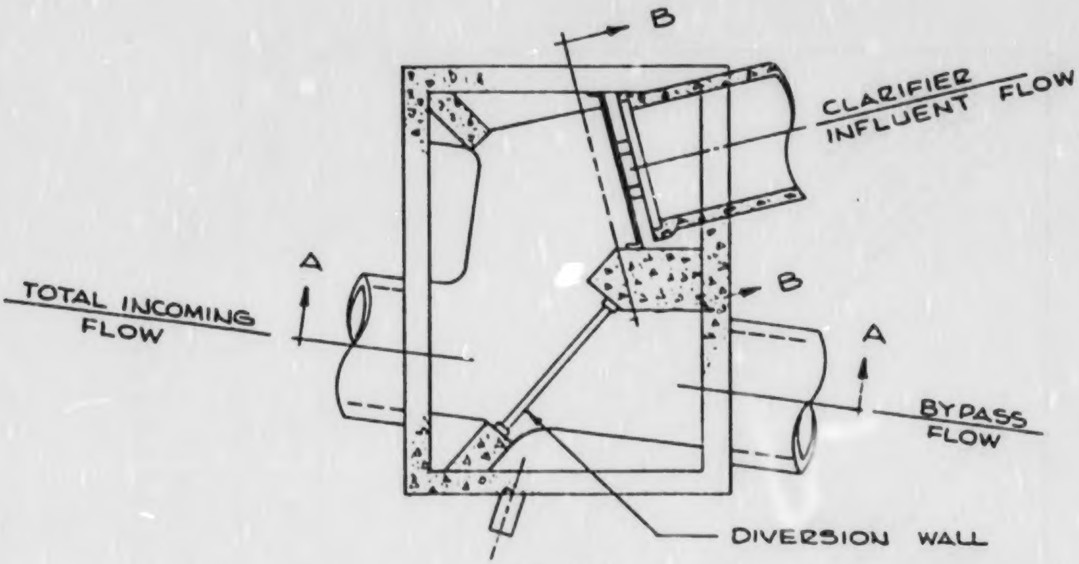
DIVERSION MANHOLE

The diversion manhole regulated, by means of an orifice and diversion wall (added during the operational phase of the project), the allowable flow rate into the clarifier. It was located at the point of connection to the existing sewer approximately 76 meters (250 ft.) west of the B & O Railroad. At this location, the combined sewer was outside the edge of the Tallmadge Memorial Parkway pavement which eliminated construction in the street and allowed for periodic visual inspections.

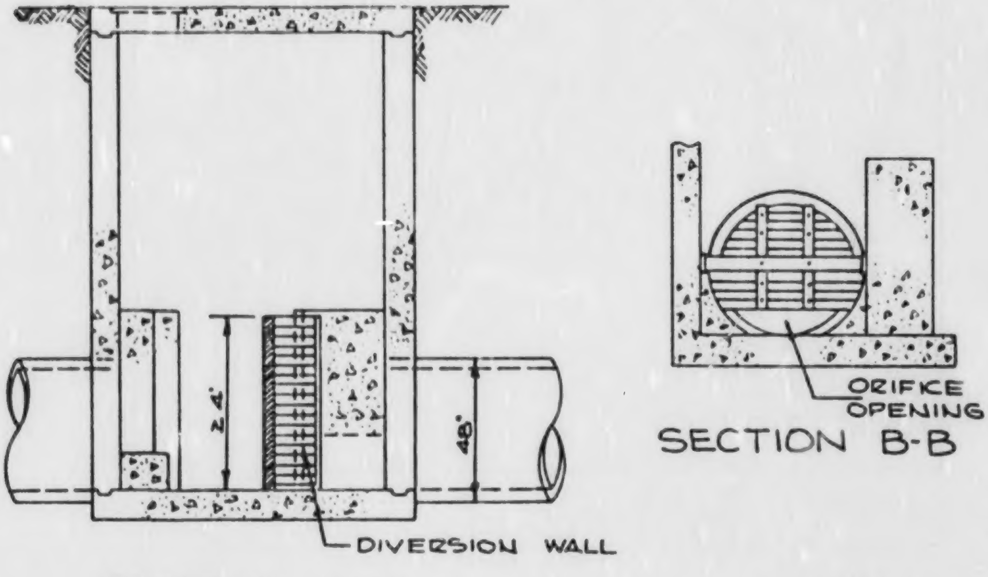
The diversion chamber, which was 2.74 meters x 3.96 meters (9 ft. x 13 ft.) rectangular and 4.27 meters (14 ft.) deep (Figure 11), diverted combined sanitary and storm flow to the storage facility. The maximum influent flow rate to the clarifier was controlled by means of the orifice and diversion wall. The height of the orifice opening and diversion wall were controlled by 8.9 centimeter (3-1/2 in.) planks of appropriate lengths. The orifice opening was adjustable from 0 to 1.22 meters (0-48 in.); the diversion wall height was adjustable from 0 to 1.83 meters (0-72 in.). A minimum height of 1.22 meters (4 ft.) was recommended for the diversion wall. This allowed for all dry-weather flow and low storm flow rates to be directed into the clarifier. During larger storm flow rates, part of the flow was bypassed into the original sewer. Figure 12 lists the theoretical total and influent flow rates expected for various orifice openings with the diversion wall height equal to 1.22 meters (4 ft.).

INFLUENT PIPE

The 1.22 meter (48 in.) diameter system influent pipe (refer to Figure 13) connected the clarifier with the diversion manhole. The culvert passed beneath the B & O Railroad tracks and was



PLAN



SECTION A-A

SECTION B-B

FIGURE 11. DIVERSION MANHOLE

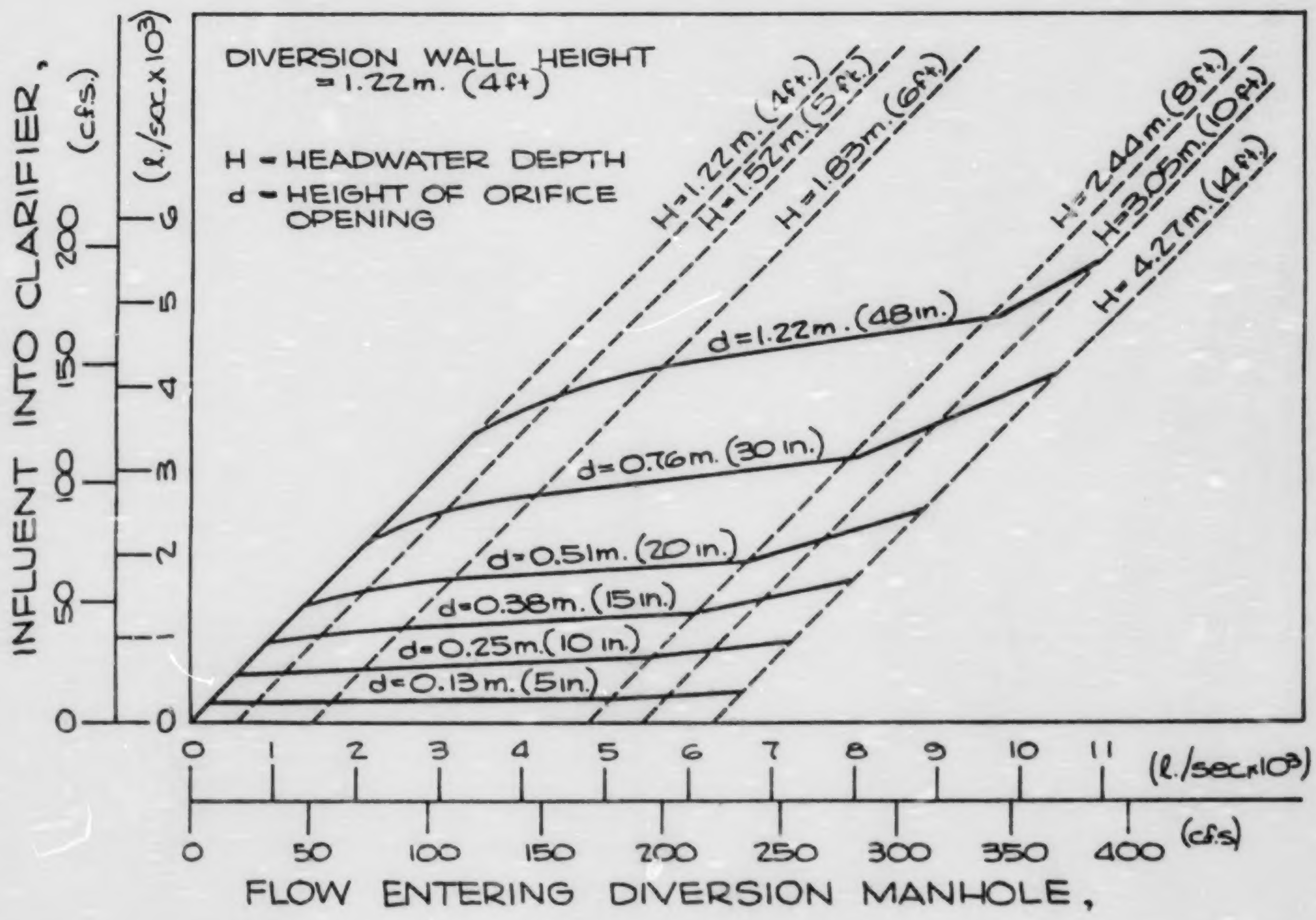


FIGURE 12. THEORETICAL INFLUENT RATES

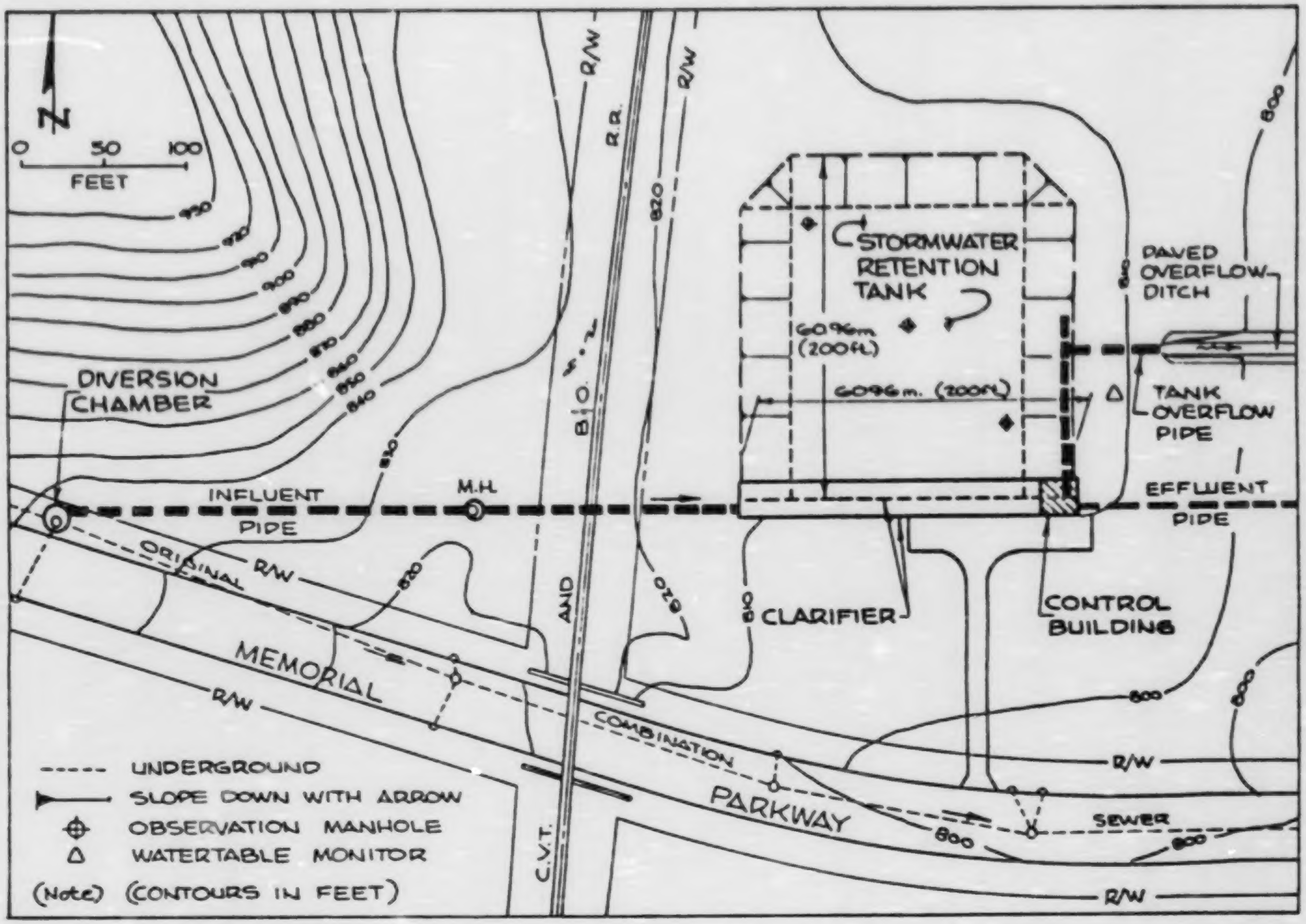


FIGURE 13. RETENTION TANK SITE

divided into two sections, each with different slopes. The first section, from the diversion manhole to the manhole approximately 21 meters (70 ft.) west of the B & O tracks, was 76 meters (250 ft.) in length with a 3.2 percent slope: the second section, which entered into the clarifier, was 40 meters (132 ft.) in length with a 10 percent slope. Given the maximum orifice flow conditions produced at the diversion manhole, the influent pipe had an ultimate theoretical capacity approaching 6510 liters per second (230 cfs).

GENERAL TANK CONFIGURATION

The general tank configuration and location was partially dictated by the site conditions. A limited land area was available due to the old sanitary landfill which borders the site to the north and east. In addition, a grey silty clay lay under the site along with a high water table above the clay. Construction in the clay strata to any appreciable depth would have required considerably greater installation costs. The allowable depth of the retention tank was, therefore, limited to 3.05 meters (10 ft.). Optimum conditions for tank depth versus cost would have required the tank to exceed 5.5 meters (18 ft.) in depth assuming acceptable soil conditions.

Figures 7 and 13 show the retention tank location and shape. The tank was a truncated pyramid with bottom dimensions of 42.67 meters x 51.82 meters (140 ft. x 170 ft.). The top dimensions were 60.96 meters x 60.96 meters (200 ft. x 200 ft.) The side slopes were constructed at a 3:1 slope due to the existing soil conditions and the bottom slope constructed at 0.25 percent slope toward the clarifier to aid in draining the cell. The south side of the retention cell consisted of, and was formed by, the sidewall of the clarifier.

Within the confines of the storage cell were three observation manholes equally spaced on a diagonal line from the southeast to the northwest corner of the site. In addition to providing observation points, the manholes housed the pressure transducers which recorded the water depth in the tank and transmitted this data to the control building. The pressure transducers were secured at a height of 7.6 centimeters (3 in.) above the bottom liner material.

The following sections describe the individual components of the retention cell and discuss their affect on the flow properties and storage capacity of the cell: liner materials, storage media, feed-overflow-draining systems.

Membrane

One of the important considerations for the storage tank to function properly was the tank liner. The following properties of liners were compared:

1. Relative cost of liner per square meter,
2. Relative cost of construction due to the number of days required for fabrication, the size of sheet available to reduce the number of field seams and to reduce potential leakage, and the ability to bond the liner to itself and dissimilar materials in the field,
3. Physical properties of elongation, puncture resistance, tear resistance, specific gravity, and flexibility,
4. Chemical resistance to environment and chemical properties of the liner material,
5. Biological resistance,
6. Maximum allowable slopes and padding requirements,
7. Relative availability, and
8. Expected life of the liner.

The types of liners studied were:

1. Natural and synthetic rubber such as (Hypalon),
2. H.C. Vinyl, pure vinyl, and P.V.C. sheet, and
3. Asphalt.

Although the asphalt concrete was the most economical liner material, the bottom tank area soil conditions (silty clay with a high water table) prohibited laying asphalt directly on the soil. An engineering study was made by the City of Akron and their consultants to determine the most feasible and least expensive liner design. The following methods were considered:

1. Placing a 15 centimeter (6 in.) concrete pad on the bottom of the tank instead of asphalt at an additional cost of $20,000.
2. Placing a 762 micron (30 mil) P.V.C. liner under the whole tank. This would have cost an additional $12,000.
3. Placing a slag base 0.30 meters (12 in.) thick over the bottom area then laying the asphalt. This would have cost an additional $20,000.
4. Placing a 762 micron (30 mil) P.V.C. liner on the bottom only, and retaining the asphalt side slopes at an additional cost of $7,148.18.

In addition to being more economical, there was the feature of testing both the P.V.C. and asphalt liner in this installation.

The original construction design of a 5 centimeter (2 in.) thick asphalt top liner material was not practical due to climatic conditions. Delays in the project, initiated by rain, postponed placing of the top liner until December, 1972. Because of the cold weather a substitution was made to a 254 micron (10 mil.) P.V.C. liner.

The recommendation for the choice of P.V.C. sheet as compared to Hypalon was made due to the following:

1. Hypalon is laid in an uncured state and gains strength during curing. Hypalon strength in a cured state is higher than P.V.C. but, maximum strength was required during construction and the Hypalon would have been at a lesser strength at this point.

2. Hypalon cures to full strength only after several months of exposure in normal use of the material. In this application, full strength may have never been realized due to the moist underground environment.

3. Hypalon could be supplied only in 18.29 meters x 69.96 meters (60 ft. x 200 ft.) sheets; thus the field seams required would have been more numerous. A P.V.C. liner could be supplied in two pieces.

The stormwater which would be stored in the tank was from a residential drainage area, therefore, the chemical properties of the effluent were not expected to promote excessive deterioration of the liner materials. The liners installed have sufficient strength to withstand the static loads and hydrostatic pressures to be encountered.

Two hundred fifty four micron (10 mil.) P.V.C. buried membranes have a twenty-five year life expectancy according to the Bureau of Reclamation.

Media

The tank media was one of the more important items studied for use in "Demonstration of Void Space Storage with Treatment and Flow Regulation". Some of the factors considered in the media study were:.

1. Void space available,
2. Physical and chemical resistence,
3. Flow of stored liquid through the media,
4. Availability in the local area,
5. Method of installation,
6. Cost per ton.

The void space available from a given media has a direct effect on tank size for a given volume, and thus, a direct bearing on all tank costs. Optimum conditions for tank costs are reached at a maximum of void space for a given media. However, maximum void space and strength to carry the load to be expected limited the type of media which could be used.

Both artificial and natural media were considered. Artificial media were (1) artificial trickling filter media, (2) tower parking media and filter blocks, (3) manufactured ceramic shapes and (4) industrial wastes or scrap.

The artificial trickling filter media did not possess sufficient structural strength for this application. The tower parking media and filter blocks were not available in the quantity necessary nor at a competitive price.

No industrial wastes or scrap were available in the area for use as media at the time of construction.

Natural media, such as bank run gravel and stone, was studied and found to be the only choice economically available in the Akron area at the present time.

The manufacture of ceramic shapes remains a possibility for future designs. The possibility of an 80 percent void space would allow the cost to be competitive even if the cost per cubic meter were twice as high as natural media. The ceramic media can be manufactured with sufficient strength for the tank media; however, installation might pose a problem.

Determination of the type of natural media to be used posed several problems:

1. What void space was available, with required washing.
2. Physical and chemical properties necessary,
3. Quantities and types of media available, and
4. Method of installation.

The specifications for the natural media chosen stated that the bulk of the media be comprised of AASHO #1 or #2 size stones with 0.102 meters to 0.152 meters (4 in. to 6 in.) of #46 stone placed on the exposed media surface. The AASHO specifications for these stone sizes are given in Table 3.

TABLE 3. SIZES OF COARSE AGGREGATES

Size No.	Amounts Finer Than Each Laboratory Sieve (Square Openings) Percentage By Weight									
	.089m 3.5in.	.076 3.0	.064 2.5	.051 2.0	.038 1.5	.025 1.0	.019 .75	.013 .50	.010 .38	NO. 4
1	90-100	-	25-60	-	0-15	-	0-5	-	-	-
2	-	100	90-100	35-70	0-15	-	0-5	-	-	-
46	-	-	-	100	95-100	-	35-70		10-30	0-5

The specifications for stone sizes #1 and #2 limited the percentages of material less than 3.8 centimeters (1.5 in.) in diameter to a negligible amount. The specific weight of the stone ranged between 1560 to 1430 kilograms per cubic meter (2630 to 2410 pounds per cubic yard).

To obtain a maximum void space, the graded material was thoroughly washed prior to placement. The porosity of the media was estimated at 40 percent, yielding a cell storage capacity approaching 3.8 million liters (1 MG). Figure 14 represents the void space volume as a function of the water surface depth in the retention tank. Solids which are deposited in the storage area during filling will reduce the porosity of the media. The extent of this problem is related to the efficiency of the clarifier, and to the amount of total solids in storm overflow. This is analyzed in the evaluation section of the report.

The chemical and physical characteristics of the media were such that the bank run type material was desirable from a void space aspect for media shape as well as strength and chemical inertness. Specifications similar to those used for trickling filter media were required to enable the media to withstand the environmental conditions to which it would be subjected.

Natural media as recommended for this system was available in the Akron, Ohio area.

Feed-Overflow-Draining System

The flow hydraulics of the storage cell are dependent upon the concepts and design of the feed, overflow, and draining systems. The feed system describes that method by which flow entered the media storage area. The overflow system describes that method by which flow exited the storage area and discharged into the Little Cuyahoga River. The draining system describes that method by which flow exited the storage area and discharged back into the clarifier.

Stormwater could enter into the media storage area by passing over the clarifier weir or by seepage through the flap gates. The rate of seepage through the flap gates, however, was determined to be minimal 28.3 liters per second (1 cfs) maximum and can be neglected. The flow over the clarifier weir and the infiltration rate into the media were a function of the developed headwater and the characteristics of the exposed media. The media characteristics of median size, gradation, porosity, and permeability were all interrelated and influenced both the infiltration and overflow flow rates. So long as fine material did not accumulate on the exposed media surface, the theoretical infiltration rate varied from 42,500 to 255,000 liters per second (1500 to 9400 cfs) depending upon the developed headwater. (Calculations for infiltration rates are presented in Appendix A.) Small sediment particles could be trapped on the exposed media surface and could drastically reduce the permeability of, and infiltration rate into, the media.

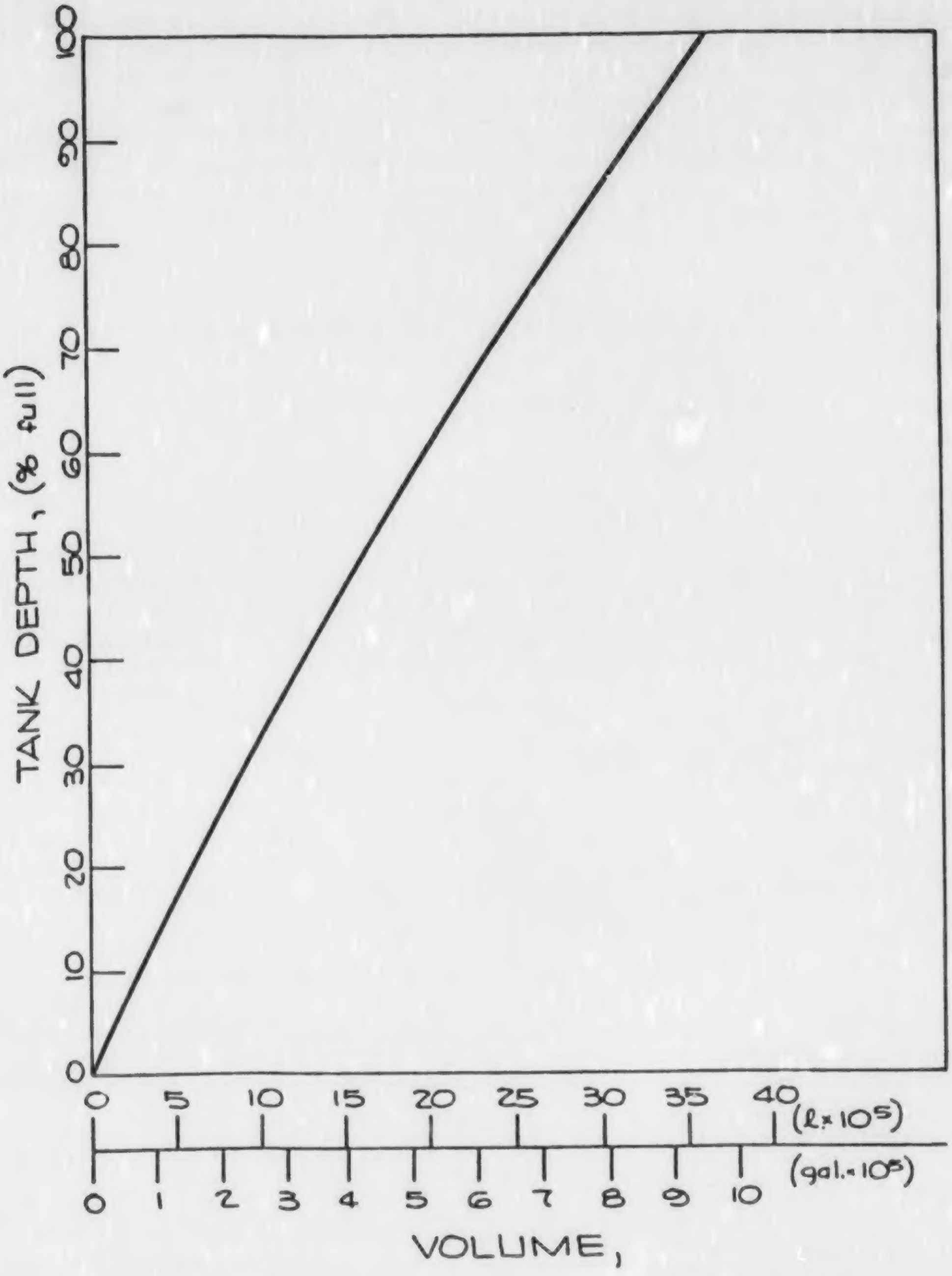


FIGURE 14. VOID SPACE VOLUME

If and when the entire media storage space was filled with stormwater, flow may exit the system by means of the 1.22 meter (48 in.) overflow risers which empty into the 1.52 meter (60 in.) overflow pipe. The overflow riser system extended from the clarifier for 32 meters (106 ft.) along the east side of the storage retention area (Figures 9 and 13). There were 12 risers protruding from the 1.52 meter (60 in.) corrugated metal pipe, one of which was located within the exposed media. The overflow pipe sloped (1/2 percent) toward a metal tee section which joined the 1.52 meter (60 in.) pipe with a 1.37 meter (54 in.) pipe. The 1.37 meter (54 in.) pipe, which had a 2 percent slope and was 15.24 meters (50 ft.) long, discharged into a concrete, trapezoidal channel. This channel, with a 1.83 meter (6 ft.) bottom width and 3:1 side slopes, directed overflow to the Little Cuyahoga River.

The overflow into the 1.22 meter (48 in.) risers occurred when the storage area was full and sufficient headwater developed on the exposed media surface. (Overflow claculations are also presented in Appendix A.) Assuming no significant accumulation of fines on the exposed media, the overflow risers could discharge approximately 4248 liters per second (150 cfs) for the maximum headwater depth of 0.91 meters (3 ft.). The overflow rate, however, was dependent upon, and limited to, the infiltration rate, therefore, accumulation of fines on the exposed media surface seriously reduced both the infiltration and overflow rates.

The draining system describes the method by which stormwater exited the storage cell and discharged through the flap gates and into the clarifier. The 1.83 meter x 1.22 meter (72 in. x 48 in.) cast iron flap gates, which were hinged to the clarifier wall, had a bottom elevation equal to that of the retention cell to allow for complete draining. There were four flap gates joining the two system components.

When a positive water surface elevation difference existed between the retention cell and the clarifier, the flap gates opened, due to the hydraulic force, and allowed stormwater to exit the storage cell. The rate of flow was dependent upon many variables; however, it was considerably greater than the maximum effluent rate of the 0.46 meter (18 in.) culvert exiting from the clarifier.

CLARIFIER

The most complex of the system components was the clarifier chamber. Within the clarifier, regulation and treatment of the stormwater was performed.

Some clarification was desirable before the combination storm and sanitary flow was diverted into the storage tank. This rectangular chamber, 60.96 meter x 1.83 x 4.72 meter high (200 ft. x 6 ft. x 15.5 ft. high), was a large volume tank 379,000 liters (100,000 gal.) designed to reduce the high influent pipe velocities and aid in the settling out of the suspended sediment. The clarifier floor was sloped at 2 percent, toward the effluent gate, to promote cleansing of the chamber during draining.

At the east end of the clarifier were two sluice gates which regulated the rate of flow exiting from the system. The flow rates from the openings were dependent upon the height of the sluice gates and the water surface elevation in the clarifier (Figures 15 and 16). The 0.46 meter (18 in.) effluent culvert directed flow to the Little Cuyahoga interceptor, whereas that of the 1.52 meter (60 in.) culvert was discharged directly into the Little Cuyahoga River. The 1.52 meter (60 in.) gate, which was designed as an emergency outlet, had a maximum opening of 0.56 meters (22 in.) allowing an overflow rate up to 2550 liters per second (90 cfs). The 0.46 meter (18 in.) culvert had a maximum flow rate of 850 liters per second (30 cfs) at 4.57 meters (15 ft.) of headwater.

Stormwater could also flow out of the clarifier by passing over the 48.77 meter (160 ft.) long weir and into the retention tank media. The flow over the clarifier weir was dependent upon the infiltration rate into the media. If sediment would accumulate on the exposed media surface, the infiltration rate and weir overflow would be drastically reduced. The overflow weir elevation was 246.26 meters (807.94 ft.) above seal level, and was 87 percent of the clarifier full level. There was 0.55 meters (1.8 ft.) of available freeboard between the weir and the cover tees. The double tees, which enclosed the top of the clarifier and the exposed media surface, were precast and were 5.11 meters x 2.44 meters (16.75 ft. x 8 ft.) with 0.05 meter (2 in.) thick flanges.

Within the clarifier were a number of subsystems which treated and evaluated the storm and sanitary flow: tube settlers, level indicator, chlorinator and samplers.

Tube Settlers

It was determined that before flow could enter the storage media, high rate clarification was needed to settle out particles from a combination sewer, given relatively short detention periods. Inclined tube settlers were designed to function as a high rate settler. A cut-out view of a settling tube module is given in Figure 17. By reducing the width of the flow paths, the wetted perimeter increases, the Reynolds number decreases, and the distance a particle is required to settle before encountering the tube wall reduces. The discrete particles which settle to the lower tube wall should agglomerate thus reducing the drag

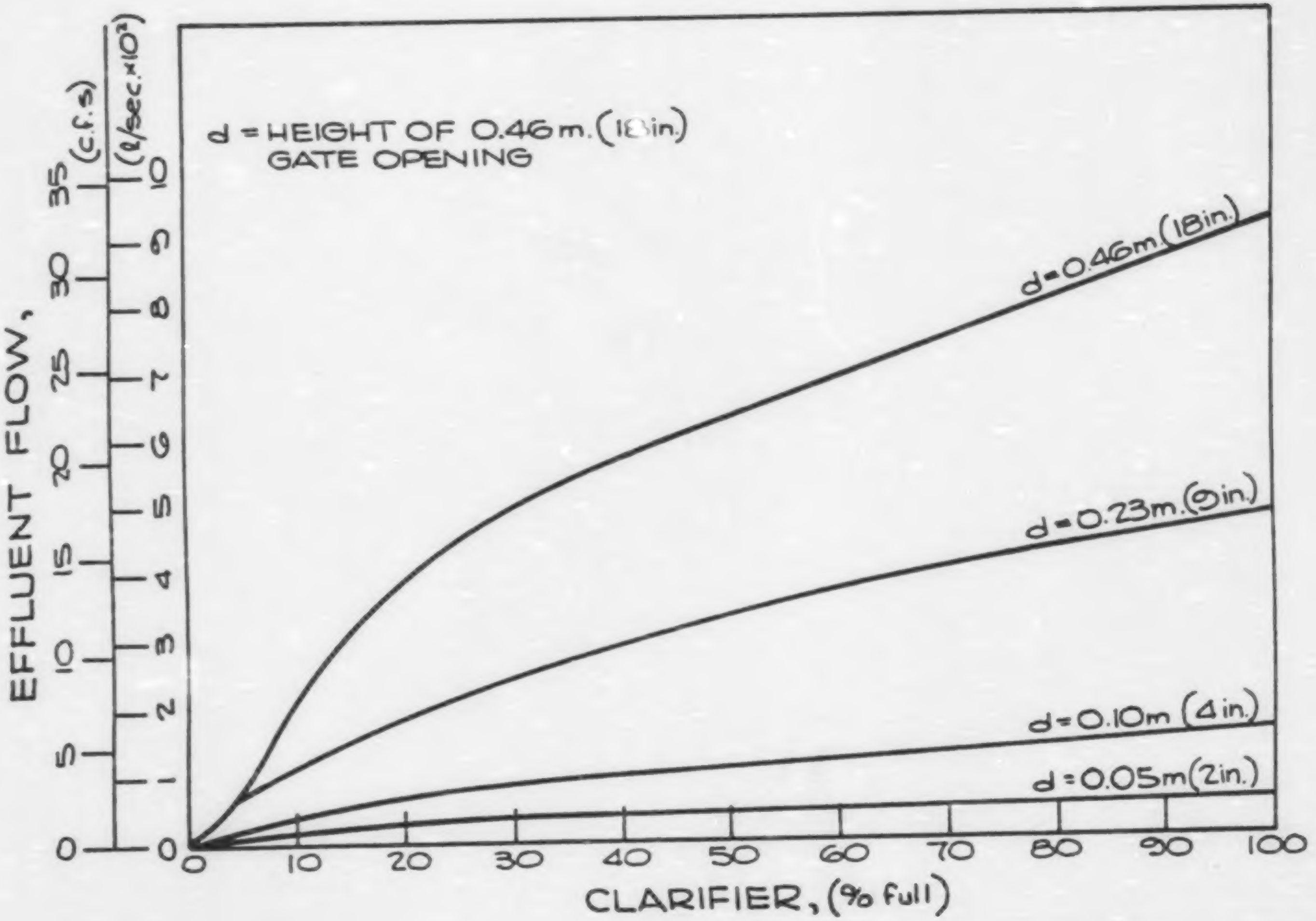


FIGURE 15. FLOW RATE FROM 0.46 METER OUTLET

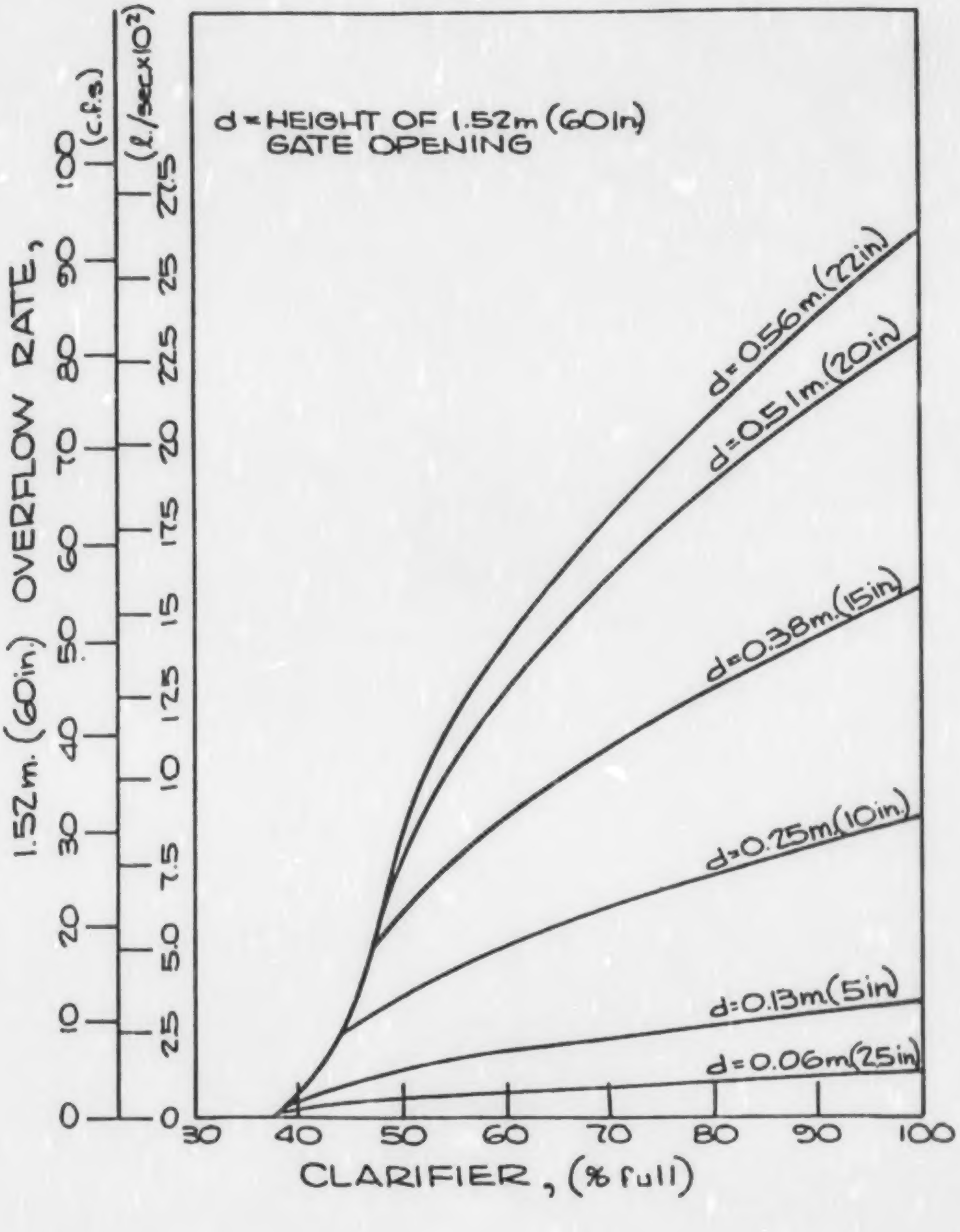


FIGURE 16. FLOW RATE FROM 1.52 METER OUTLET

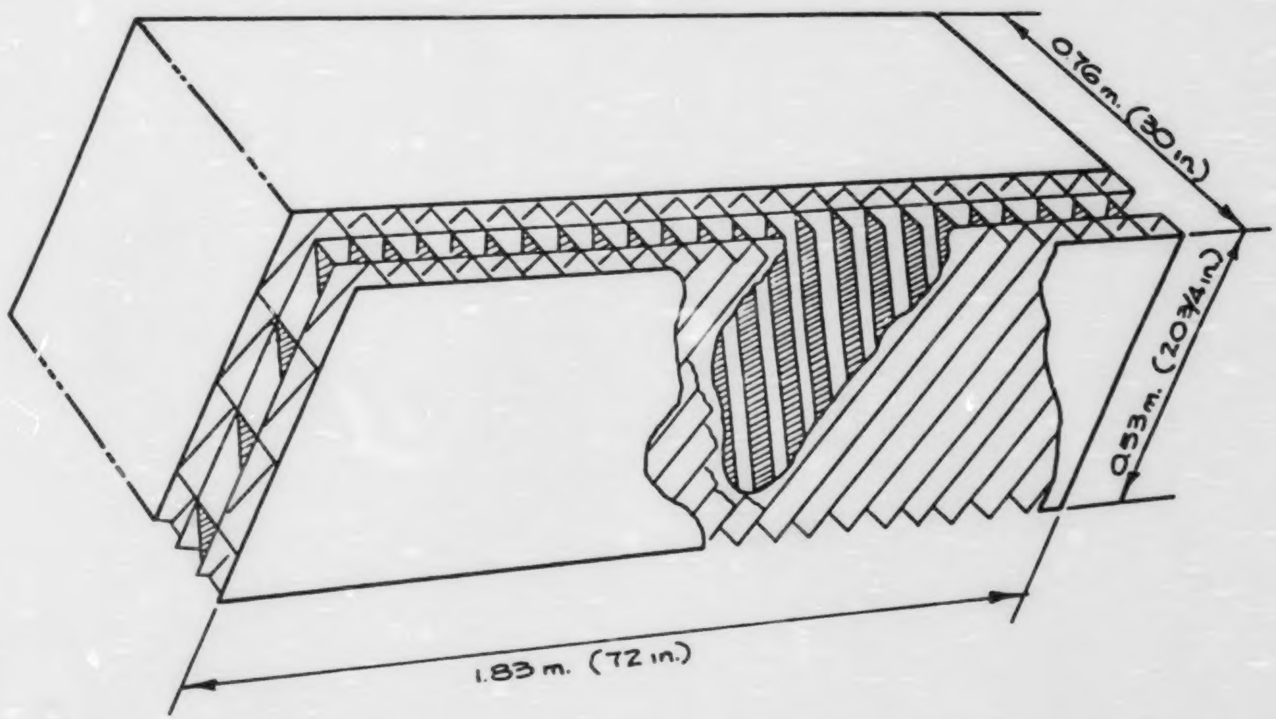


FIGURE 17. CUT-OUT VIEW OF SETTLING TUBE MODULE

resistance exerted against them. This reduced drag resistance may allow the agglomerated particles to slide down the tube wall and settle to the bottom of the clarifier, thus reducing the carryover of settleable material from the flow passing over the clarifier weir and onto the exposed media surface. The settling tube modules may, incidentally, reduce the carryover of suspended solids.

The manufacturer's operating instructions manual commented that the installation of settling tube modules would not automatically improve the performance of the settling basin. The performance of the tube modules is a function of the following parameters; basin inlet hydraulics, basin effluent collection hydraulics, water temperature, chemical feed and coagulation, flocculation, influent characteristics and range of variations, floc settling rate, sludge removal capability and effectiveness, flow rate change frequency and duration, filter design, maintenance and cleaning, and operator ability and dedication. Many of these parameters apply to the retention tank system.

Approximately 85.47 square meters (920 sq. ft.) of settling tube modules were installed in the clarifier to reduce the carryover of settleable material. The 0.05 meter (2 in.) square tubes were inclined at a 60° angle from the horizontal and had a flow length of nearly 0.61 meters (24 in.). The tube modules were installed at a bottom elevation of 245.4 meters (805.1 ft.) above sea level (69% of clarifier height).

Clarifier Level Indicator

The clarifier influent flow rate and water level were measured by evaluating the recorded data of the purge air bubbler type depth measurement system. Elevations of important structural components can be represented as corresponding percent of clarifier full height (Figure 18). Air was forced, by a 5.07 HP (5 HP, U.S.) air compressor, through the 0.64 centimeter (1/4 in.) piping of the bubbler mechanism which extended vertically to the clarifier bottom near the 0.46 meter (18 in.) gate valve. The compressed air was discharged into the line at a slow constant rate which, by eliminating silt buildup, was self-cleansing.

When the water level in the clarifier exceeded the elevation of the outlet of the 0.64 centimeter (1/4 in.) line, (7.6 centimeters or 3 in.), the static pressure produced by the water elevation resisted the flow of air in the pipe. Measurement of the pneumatic pressure was analogous to the water level in the clarifier and was converted by an electronic differential pressure transmitter to give a specific chart recorder pen response.

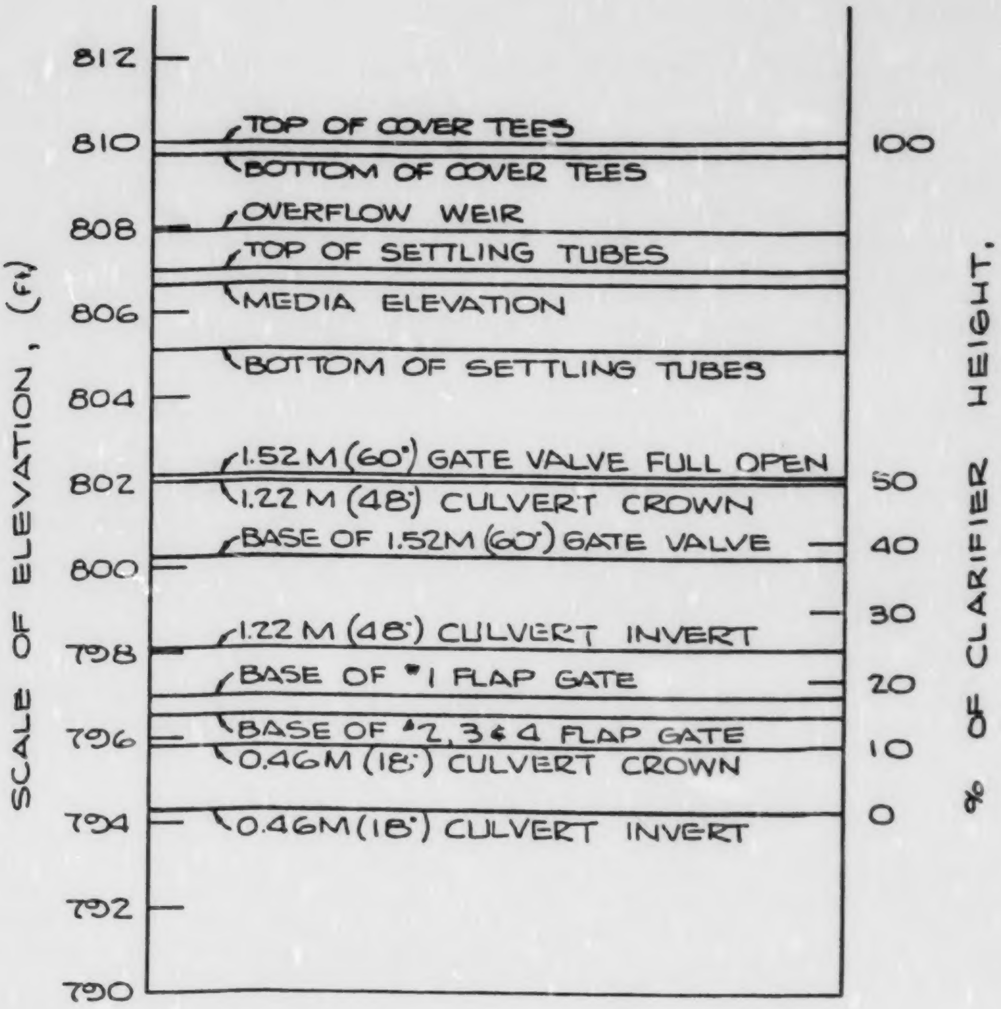


FIGURE 18. REPRESENTATIVE COMPONENT ELEVATIONS

The single purge air bubbler had three pressure transmitters which yielded water level recordings from 0 to 0.508, 0 to 4.78, and 4.22 to 4.78 meters (0-20, 0-188, and 166-188 inches). The bubbler mechanism and the 0 to 0.51 and the 0 to 4.78 meter (0-20 and 0-188 in.) level recorders operated continuously. The 4.22 to 4.78 meter (166-188 in.) level recorder was energized when the clarifier level reached 4.22 meters (166 in.). Relavs on the different transmitters controlled the annunciator alarm lights, sampler operation, chlorine pump, and colorimetric chlorine analyzers.

The system's average influent and effluent flow rates could be calculated through the integration of the clarifier water level with respect to time and gate openings. The volume of stormwater contained within the clarifier was dependent upon the water surface level as represented in Figure 19.

Chlorinator

The Tallmadge Memorial Parkway Retention Tank #1 was designed not only to detain and regulate the peak storm flows but also to chlorinate that flow which was stored in the media or was overflowed to the Little Cuyahoga River. A proportional volume of 15 percent sodium hypochlorite solution was used to disinfect the stored combined sanitary and stormwater. Chlorine was added to the stormwater entering the system at the influent pipe when the water surface elevation equaled that of the clarifier weir.

Three 5678 liter (1500 gallon) subsurface storage tanks were located near the southwest corner of the clarifier. A 190 liter per minute (50 GPM) pump drew the sodium hypochlorite solution from the tanks and discharged a portion of it into the clarifier at the outlet of the 1.22 meter (48 in.) influent pipe. The remainder was pumped back into the storage tanks. The quantity of the solution pumped to the clarifier was regulated by a manually preset ratio controller.

Both the sodium hypochlorite pump and the ratio controller were dependent upon the water surface level in the clarifier. At a clarifier water level of 85 percent of full (4.06 meters or 160 in.) the sodium hypochlorite pump was triggered and began to prime. At 87 percent of full (4.22 meters or 166 in.) the preset ratio controller was activated. The ratio controller could be preset between 0.3 to 3.0 which, in conjunction with the clarifier water level, regulated a 4 to 20 ma D.C. output signal. Given any preset ratio value, the output signal increased lineally with increasing clarifier water levels above 4.22 meters (166 in.). The 4 to 20 ma D.C. output signal was converted into an 0.21 to 1.06 kilogram per square centimeter (3 to 15 psi) air signal by an electric pneumatic transducer. The air signal adjusted the diaphram controlled pinch valve located directly

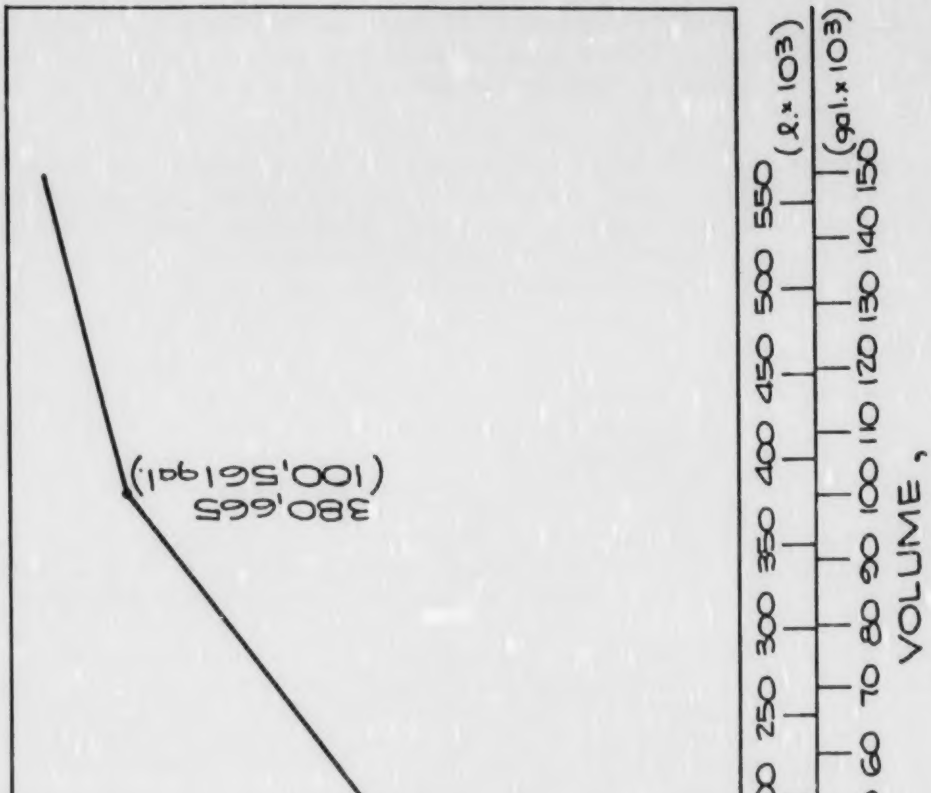


FIGURE 19. VOLUME IN CLARIFIER

after the 190 liter per minute (50 GPM) pump, which regulated the flow of hypochlorite solution to the clarifier.

The chlorine residual of the stormwater was monitored at the tank influent, within the tank, and the tank overflow by analyzing an aliquot obtained by the automatic samplers. The chlorine residual was measured colorimetrically by two Hach CR2 automatic chlorine residual analyzers. The data from the analyzers was relayed to the control panel recording charts. If the detected chlorine residual was improper, the ratio controller could be adjusted accordingly. The ratio controller was housed in the analog control center cabinet. (Refer to Figure C-1).

Samplers

Three automatic samplers were incorporated into the project to analyze the combined sanitary and stormwater flow. The samplers could also be operated manually. The inputs for the samplers were installed at four locations: adjacent to the outlet of the 1.22 meter (48 in.) influent pipe, above the tube settlers prior to flow over the weir, within the 1.52 meter (60 in.) overflow pipe, and in the retention tank observation manhole #1. Sampler #1, which collected from the influent flow, activated when the clarifier water level reached 87 percent of full (4.22 meters or 166 in.). The sampler turned off when the water level dropped below 4.22 meters (166 in.). Sampler #2 collected samples from both the flow above the tube settlers and from observation manhole #1. Within the control building, on the north wall, were two gate valves which had to be manually regulated to obtain the desired sampling location. The sampler automatically started when the flow exceeded 4.22 meters (166 in.) in the clarifier and stopped when the water level in manhole #1 dropped below 7.6 centimeters (3 in.) in depth. Sampler #3, which collected from the 1.52 meter (60 in.) overflow pipe, activated when the depth of flow in the 1.52 meter (60 in.) pipe reached 7.6 centimeter (3 in.) and turned off when the depth of flow dropped below 7.6 centimeter (3 in.).

When activated the samplers (Rohrer Model 2) ran continuously. An isometric representation of the automatic sampler is shown in Figure 20. Cam timers were incorporated into each sampler and were adjusted to discharge one 1.9 liter (1/2 gal.) sample every 5 minutes. A stepping motor rotated the funnel feed system to another sample bottle after each 5 minute interval. The automatic samplers had a capacity of 24 samples. After filling all 24 bottles, the sampler automatically stopped. If additional samples were required, the bottles were changed and the sampler reactivated. A small composite sample was also taken every interval and stored in an 18.9 liter (5 gal.) capacity jug. All additional flow from the samplers was drained back into the clarifier.

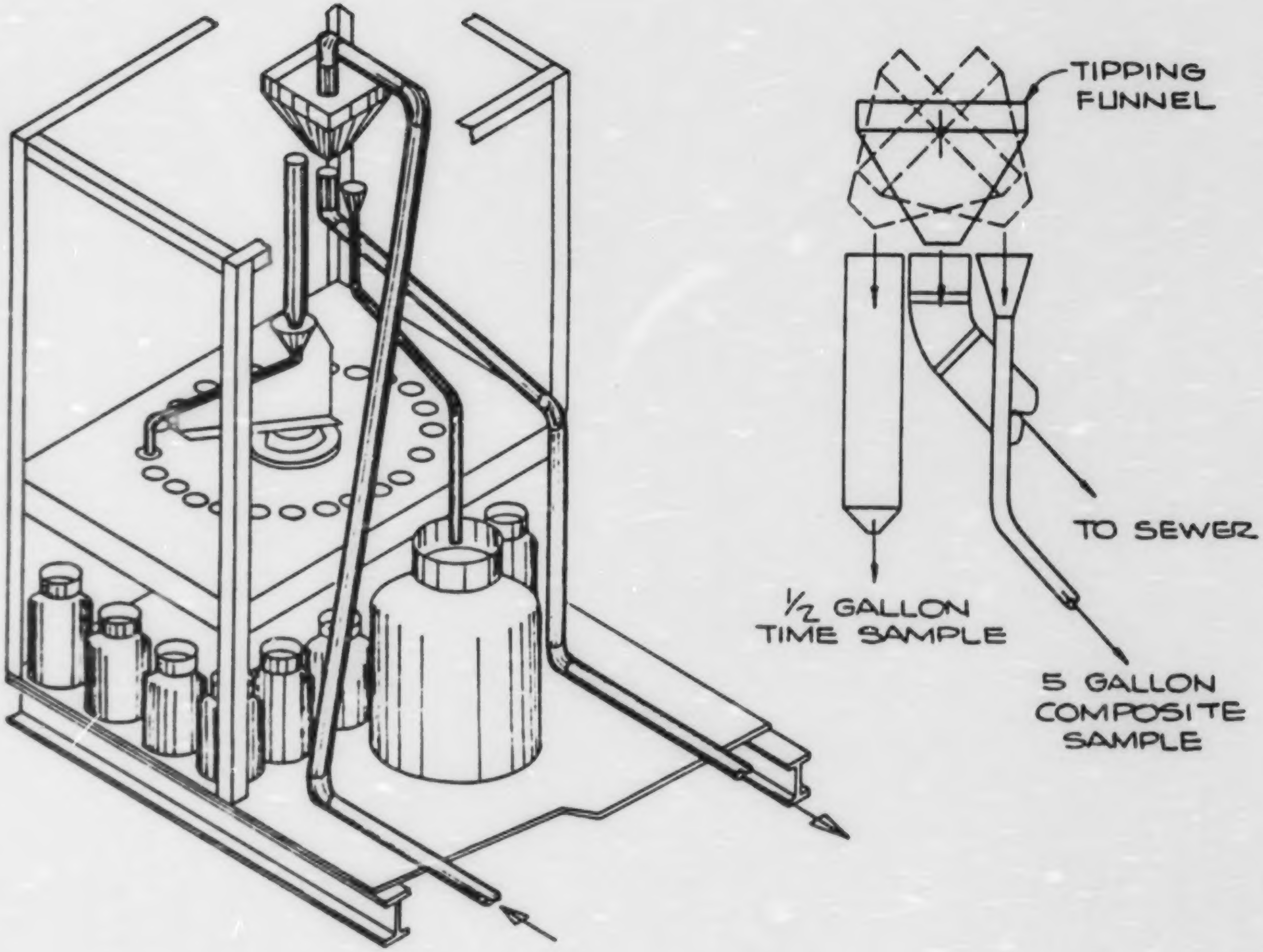


FIGURE 20. ISOMETRIC OF AUTOMATIC SAMPLERS

TANK LEVEL INDICATORS

The water surface elevation in the media storage area, retention tank, was monitored by three differential pressure diaphrams, one in each of the three observation manholes. The pressure diaphrams were secured to the side of the manhole approximately 7.6 centimeters (3 in.) from the bottom liner material.

When the water level in the retention tank exceeded the elevation of the bottom of the diaphram the static water pressure signal was transmitted from the diaphram to a differential pressure transducer. The transducer output a 4 to 20 ma D.C. signal which initiated a chart recorder pen response.

Each of the three pressure diaphrams had a separate chart recording. The retention tank water level chart recorders ran continuously.

GROUND WATER MONITOR

On the east side of, and adjacent to, the retention tank was a purge air bubbler device installed in a well casing to monitor the ground water level. Similar to the purge air bubbler in the clarifier, the pressure signal was converted to an electrical signal which was transmitted to the control center recorder. The bubbler measured the ground water level from an elevation 0.61 meters (2 ft.) below the bottom of the retention tank to an elevation equal to that of the exposed media. The continuous chart recorder measured the 0 to 5.08 meter (200 in.) range on a 0 to 100 percent full scale readout. The ground water meter was installed to check the condition of the retention tank media. If a rupture in the liner material would occur during filling, a sharp rise of the ground water level would occur.

RAIN GAUGE

Due to isolated thunderstorms and the variability of rainstorms over large areas, it was deemed necessary to incorporate a rain gauge at the testing facility. A Belfort Instruments (5915 series) recording rain gauge was installed within the control building. Precipitation was collected from a funnel on the roof and transferred to a galvanized bucket mounted on a weighing mechanism. The deflection of the weighing mechanism produced by the weight of the precipitation was transferred through a gear and lever mechanism to a low torque, 25,000 ohm precision potentiometer.

The rain gauge mechanism had an accuracy to 0.13 centimeters (0.05 in.) and the data was plotted on a continuous chart recorder which measured 0 to 0.25 meters (0 to 100%) full scale.

SECTION VIII

TOP ACTIVITY

The Tallmadge Memorial Parkway Stormwater Retention Tank #1 is a novel concept in underground storage and provides a top surface usable for other purposes. Possible usages would include parking areas, sporting facilities or playgrounds which could be accommodated.

Future plans for the city property at the site area were for recreation or park land along the Little Cuyahoga River Valley. In order that the finished tank area may be compatible with its location, the Akron Parks and Planning Departments were consulted about the Project. No specific plans had been completed for the immediate site area.

The neighborhood west of the site area is generally above average in income levels, while the neighborhood to the east is below average. To the north is additional city owned property and to the south is a sparsely developed are in the valley.

Both neighborhoods, east and west would require a travel distance of at least one-half mile to reach the site. The Tallmadge Memorial Parkway is a busy thoroughfare which presently has no sidewalks along the steep grades up both sides of the valley. Therefore, it was felt that an adult type playfield would be most compatible with the conditions.

As in most cities today, Akron is deficient in recreation areas and is in a program to improve this condition. Additional baseball and softball facilities are needed at present, which would be compatible with the underground tank facilities.

SECTION IX

CONSTRUCTION AND EVALUATION

Construction of the prototype facility was begun in July 1972 by the Bay Construction Inc., of Sandusky, Ohio. Photographs of the construction of the prototype facility are presented in Figures 21 through 23. Figure 21a is a view looking north at the installation of the asphalt side liner. In the foreground is the overflow riser system. Figures 21b and 22a show the placement of the P.V.C. bottom liner material. In the background of Figure 21b is the clarifier's north wall with openings for the flap gates. Figure 22b shows the backfilling of the storage cell with media. A close-up of the AASHO #1 and #2 size aggregate is in the foreground. The square perforated concrete columns in photograph 22b, are the observation manhole casings. Figure 23a is a view looking west at the corner of the control building and the top 254 micron (10 mil) P.V.C. liner. Figure 23b looks east at the clarifier and exposed media surface. Note the placement of the precast concrete cover tee sections.

At the bottom excavation grade for the retention tank, saturated grey silty clay was encountered as indicated by the soil borings. Four French drains were installed in the bottom of the tank excavation to drain away ground water and dry the grey silty clay material. The French drains partially aided in dewatering the silty clay, however, the existing soil condition was not suitable for the placement of asphalt directly on the soil. An engineering and economic study was made, and a 762 micron (30 mil) P.V.C. liner was substituted as the bottom material.

Excessive rain throughout October and November caused considerable delays in construction. These delays necessitated the redesign of two system components. The approaching cold weather made the asphalt top liner material impractical, therefore, a substitution was made to a 254 micron (10 mil) P.V.C. liner. The overflow ditch, which was originally to be paved with asphalt, was instead paved with reinforced concrete.

Delays in the shipment of vital equipment needed in the operation of the clarifier section postponed the completion date of the facility. The Armco flap gates and sluice gates for the clarifier were not received until the end of December 1972. In addition, the 84 square meters (900 sq. ft.) of clarifier tube

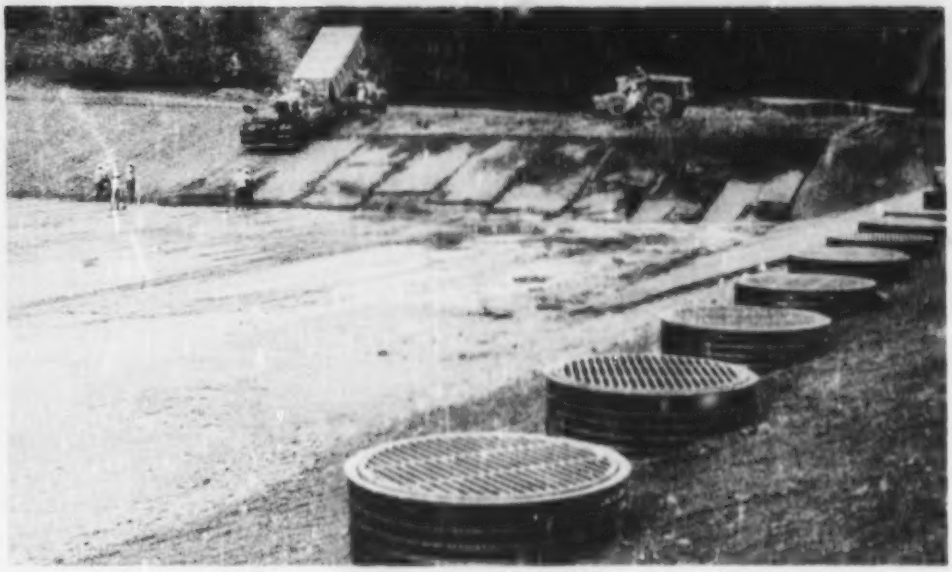


FIGURE 21a. INSTALLATION OF ASPHALT SIDE LINER

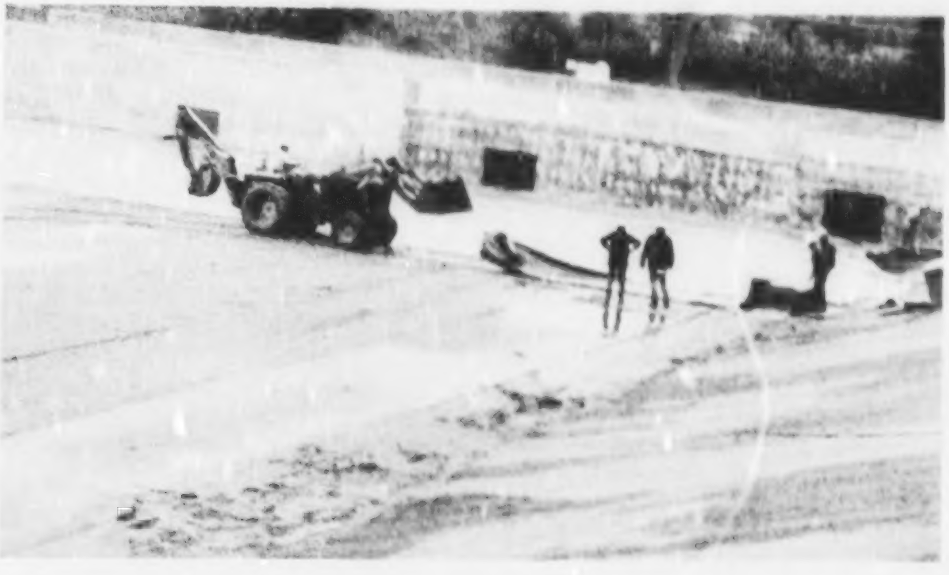


FIGURE 21b. INSTALLATION OF BOTTOM LINER (WEST)

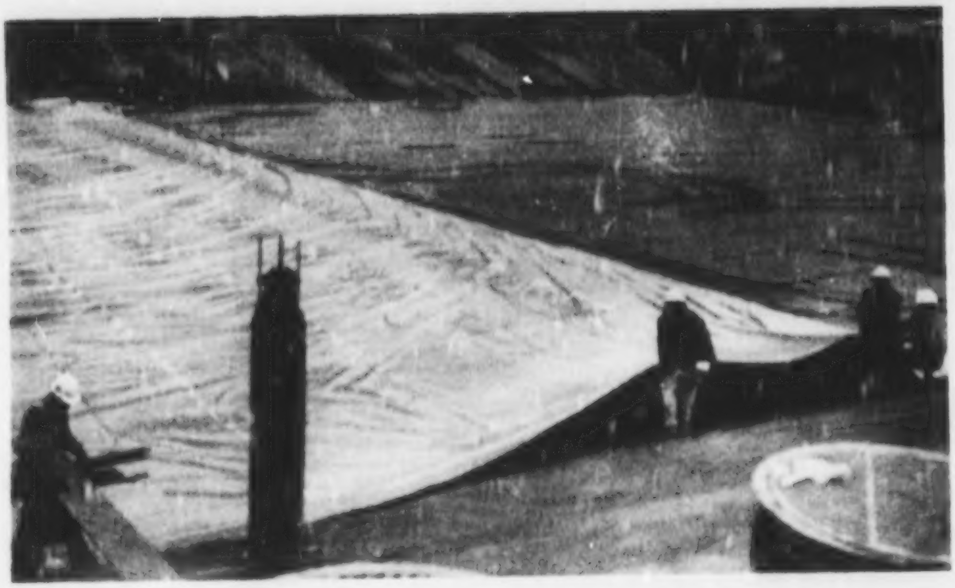


FIGURE 22a. INSTALLATION OF BOTTOM LINER (SOUTH)

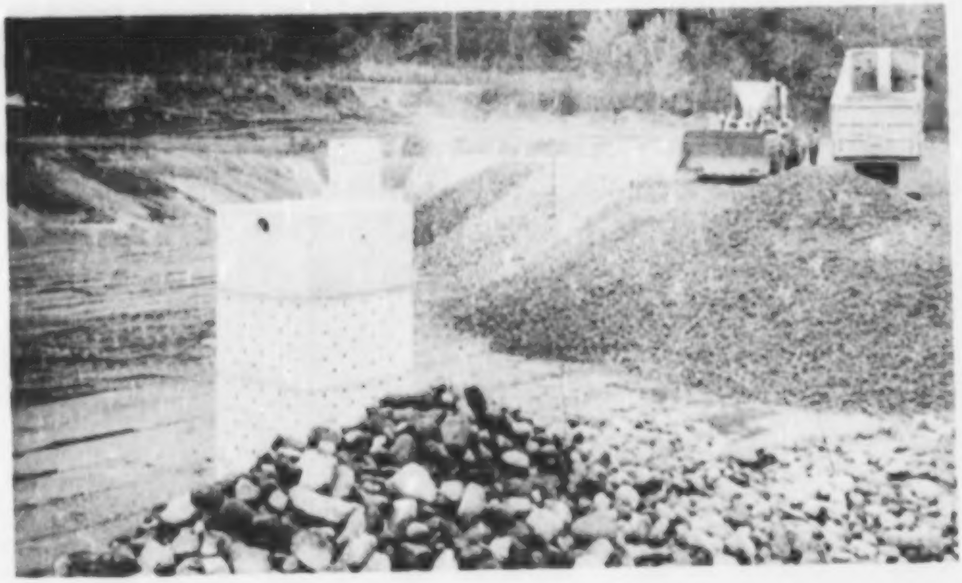


FIGURE 22b. BACKFILL WITH MEDIA

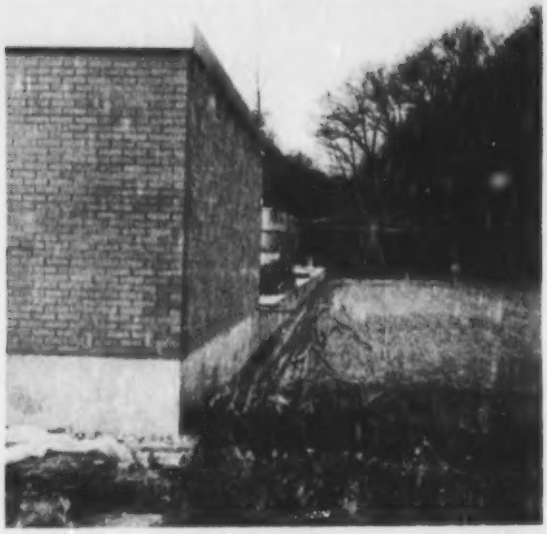


FIGURE 23a. TOP LINER MATERIAL

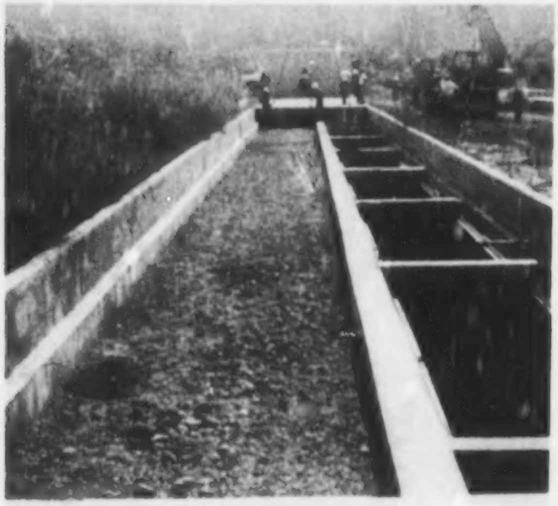


FIGURE 23b. CLARIFIER AND EXPOSED MEDIA

settlers, when delivered to the site, were inspected and found to be unacceptable. Field repairs to the modules would have been insufficient, therefore, these modules were not used. A new supplier was chosen and their tube settlers were installed in the clarifier.

A problem developed during construction with sewer gas backing up through the 0.46 meter (18 in.) effluent pipe from the Little Cuyahoga Interceptor to the clarifier. A one-way gas valve was installed in the first manhole above the interceptor sewer which eliminated the problem.

On January 1, 1974, the operational phase of "Demonstration of Void Space Storage with Treatment and Flow Regulation" was initiated. The two year operational period of the prototype facility tested the applicability of the void space storage "Geo-Cel" concept.

The existing combined sewer system for the Tallmadge Memorial Parkway drainage area was designed to overflow into the Little Cuyahoga River when the flow at the interceptor exceeded 278 l/sec (9.8 cfs). Based on the Rational Formula for flow computation, a rainfall with an intensity greater than or equal to 0.08 cm (0.033 in.) per 20 minute duration would be sufficient to cause overflow to the river. Approximately one hundred and fifty (150) precipitation events, exceeding the minimum intensity promoting overflow, occurred during the two year operational period. These precipitation occurrences, with related operational data, are listed in Appendix D, Table D-1. Of the 150 events that would have promoted overflow at the original interceptor device (9.8 cfs), twelve selected storms filled the clarifier to the weir level. Automatic and grab sampling data was obtained during these storms and is presented in Appendix D Tables D-2 and D-3.

During the operational period, several unusually severe storms, in addition to mechanical malfunctions, prevented acquisition of a considerable amount of data. One of the first significant storms occurred on April 1, 1974, when a seventy-five year intensity downpour beset the retention tank. The recording rain gauge, which collects rainfall through a funnel on the control building's roof, indicated a rainfall of 5 cm (2 in.) in a duration of 25 minutes. The deluge caused considerable erosion of the steeply sloped (9%) drainage area and carried heavy concentrations of inorganic solids into the system.

The flow in the original combined sewer was intercepted at the diversion manhole and a majority of the storm flow was diverted into the clarifier. The clarifier filled to capacity in 20 minutes at an average flow rate of 935 l/sec (33 cfs). The clarifier filling rate (33 cfs) was determined through the integration of the clarifier water level surface with respect to time. It was anticipated that the flow entering the clarifier increased with time up to maximum flow rate of 6230 l/sec (220 cfs) as calculated by a combination of the Rational Formula and Figure 12.

Due to the high flow rates entering the system, the large concentration of solids produced by the eroded slopes was not reduced by the tube settlers and was partially deposited on the surface of the exposed media. When solids are trapped on the exposed media surface the infiltration rate into the media can be reduced. (Calculations are presented in Appendix A.) The stormwater that could not enter the void space storage cell moved several of the clarifier cover sections (a few inches) and flowed overland to Memorial Parkway. The precast concrete cover sections, which were not anchored down, were designed as an emergency overflow to provide water pressure relief during extreme flow conditions.

The storm flow notification system malfunctioned, therefore, personnel were not at the site in time to manipulate the gate valves, which would have alleviated the extreme buildup of pressure in the clarifier. The notification system, which consisted of a copper pronged probe suspended inside the clarifier at a specified elevation, sent a signal to the Akron Sewer Maintenance Building when the clarifier water level reached the probe elevation. The signal illuminated a red light on their control panel, and the dispatcher, upon recognition of the illuminated light, notified personnel from Karl R. Rohrer Associates, Inc. When notified, between 20 to 30 minutes travel time was required for personnel to arrive at the site. For high intensity, short duration storms, the notification system was not adequate since the time of concentration (that time required for water from the most remote point in the drainage area to reach the facility) for the drainage area was only approximately 20 minutes.

Full operation of the prototype facility was not resumed until October 1974, when repairs and re-evaluation studies were completed. After removing the cover tee sections, a layer of sand was observed ranging from 15 cm (6 in.) in depth at the point of flow over the weir, to 2.5 cm (1 in.) at the north edge of the exposed media (8.25 ft. width), and extending for the entire exposed media length (200 ft.). The surface silt, sand, and grit deposits were removed from the exposed media surface, two masonry seals in the clarifier entrance chamber were replaced, the clarifier cover sections were reset, and a manually preset throttle orifice in the diversion manhole was designed and installed. Particular attention was given to the design of the diversion manhole throttle orifice which limited the maximum flow rate allowed to enter the clarifier. The orifice opening was adjusted to allow only that flow rate into the system which could be drained out through the overflow and effluent gates. All additional flow was diverted back into the original sewer.

Reassessment of the physical limitations of the system, combined with the critical drainage area conditions, dictated a conservative approach in the testing of the retention tank. To prevent any excessive storm flow from entering and possibly restricting the infiltration rate into the exposed media, the overflow and effluent slide gates were adjusted to allow all dry weather and stormwater flow to pass through the clarifier without causing flow over the weir until an observer arrived on the site. The observer, upon notification or anticipation of a precipitation event, would travel to the site and make the appropriate gate adjustments to allow storm flow to enter the media. This testing procedure was deemed necessary to prevent events similar to that of the April 1, 1974 storm from occurring. The detrimental effects produced by sediment reducing the infiltration rate into the exposed media surface illustrates the necessity of a fail-safe storage cell feed system in future installations. This could possibly be accomplished with one or more of the following: an automatic overflow system which would prevent excessive pressure buildup given a reduction in the infiltration rate, a well feed system which would be partially self-cleaning during draining of the cell, or a media with greater void space and less surface to surface contact points (possibly an artificial media) which trap sediment and reduce infiltration rates.

A general investigation of the prototype facility indicates that with these and other minor design changes, the void space storage concept could be a practical means for the prevention of untreated combined, or storm, sewer overflows to receiving water bodies.

It is also envisioned that future installations would be designed to operate automatically (unattended) and without the sophisticated instrumentation which was incorporated in the prototype facility.

No noticeable offensive odors or flammable gases (detected by the combustible gas monitors) were generated in the storage area during the two year operational period. A combination of factors were responsible for this: the storm flow from this steeply sloped combined sewer system consisted mainly of suspended inorganic solids rather than putrescible organic materials, chlorination of the flow entering the media prevented the wastewater from going septic, and the detention times of the stored stormwater were relatively short. Slime and algal growth within the media were not visually detected, and there were no rodent or insect infestations.

The four clarifier flap gates, which allowed the stored stormwater to flush by gravity back into the clarifier, did not completely seal and permitted minor seepage (1 cfs maximum) to circumvent the tube settlers and enter the void space storage tank during filling. The effluent and overflow slide gates were also observed not to seal as tightly as expected, allowing minor quantities of flow to escape the clarifier during filling events.

The underground P.V.C. and asphalt lined storage tank "Geo-Cel" performed satisfactorily. No membrane ruptures or surface infiltrations were detected. No subsidence of the ground surface level above the tank, which would indicate a rupture of the top P.V.C. liner, was observed. Also, no unusual increased groundwater levels occurred on the groundwater chart recorder during filling of the retention tank, which may have indicated a side or bottom membrane rupture.

CLARIFIER WATER LEVEL INDICATOR

The nucleus of the testing and evaluation of the facility was the single purge air bubbler which was located adjacent to the 0.46 meter (18 in.) effluent gate. Measurement of the pneumatic pressure in the bubbler was analogous to the water level in the clarifier and was converted by an electronic differential pressure transmitter to give a specific chart recorder pen response. Activation of the annunciator alarm lights, automatic samplers, chlorine pump, and colorimetric chlorine analyzers were dependent upon the electronic relays of the single purge air bubbler. In addition, approximate influent and effluent flow ratew were calculated through the integration of the clarifier water level with respect to time and gate openings (Figures 15 and 16). These calculations gave only the average flow rate for a specific time interval, based on the recorded data of the single purge air bubbler. Malfunctions of the air compressor, used to supply air to the bubbler mechanism, electronic drift in the switch relays, and loss of inking in the chart recorders caused minor loss of operational data.

The evaluation of the automatic samplers, tube settlers, and chlorination system will be discussed individually, although they are interrelated and dependent upon the purge air bubbler for activation.

AUTOMATIC SAMPLERS

The feasibility evaluations of two system components were primarily dependent upon the data obtained by the three automatic sampling devices which were incorporated into the system. Correlation and comparison of the sampling data was to be used for a general evaluation of the tube settlers and the media void space storage concept. The effectiveness of the tube settlers was to be predicted by the differences in solids between the samples taken with the influent sampler and the weir sampler. The weir sampler data was to be used to estimate the volume of solids carried into the retention media, while the observation manhole sampler data, combined with that obtained with the weir sampler, was to be used to analyze the leaching of solids through the media.

Although a number of grab samples were taken during dry-weather flows and precipitation events, the bulk of the sampling data was from the automatic samplers. The collection pipes for the three automatic samplers were installed at four locations: adjacent to the outlet of 1.22 meter (48 in.) influent pipe, above the tube settlers prior to flow over the clarifier weir, in the retention tank observation manhole #1, and within the 1.52 meter (60 in.) overflow pipe. A detailed description of the sampler operation is presented in Section VII of this report.

The twelve precipitation events which activated the influent and weir samplers were analyzed by some or all of the following tests: total solids, suspended solids, volatile solids, settleable solids, BOD, COD, pH, ammonia, nitrite, nitrate, and phosphorous. The results from these tests are presented in Appendix D, Tables D-2 and D-3.

Correlation of the sampling data was difficult to access since the influent sampler activated only after the clarifier had filled with stormwater to the point of flow over the wier and into the exposed media surface. The influent sampler did not collect the anticipated first flush of suspended solids or BOD. The first flush concept associated with combined sewers is generally characterized as the large concentrations of SS and BOD which are collected at the inception of a storm, due to the settleable material which had been deposited within the sewer during the dry-weather period prior to the storm. Due to the activation relay of the influent sampler, any first flush of solids or BOD had already passed the sampler's collection point and was contained within the clarifier. It was impractical, however, to change the influent sampler's activation relay point since an evaluation of the first flush concept at the clarifier entrance would have required the sampler to run continuously. Analysis of the weir sampler data does indicate high initial concentrations of suspended solids, however, it is predicted that this was due to that sediment which was carried into the inlet basins as runoff from the steep drainage area (Refer to Table D-2 in Appendix D). From the sampling data obtained, the concentration of suspended solids ranged from a high of 9,800 mg/l, to a low of 13 mg/l, and with an average of 510 mg/l. Decreasing concentrations of suspended solids were evident with increasing durations of time from the initiation of a storm. The weir sampler data does not indicate a severe first flush of BOD which would be produced by the buildup of organic solids within the sewer during dry-weather flows. Since the Tallmadge Memorial Parkway drainage area is steeply sloped (9%), the organic material does not settle out within the sewers during dry-weather flows, therefore the first flush concept for this particular drainage area was not evident and thus may not be representative of a typical combined sewer system.

The automatic tank sampler activated on only four precipitation events and the overflow sampler activated only once. Some of the sampling data was lost due to compressor and sampler malfunctions.

TUBE SETTLERS

An in-depth evaluation of the tube settler efficiency is not possible based on the data obtained by the automatic samplers. The direct reduction of suspended solids by the tube settlers cannot be calculated because of the influent sampler's location and activation relay point within the purge air bubbler (level indicator). Upon initiation of a storm a high initial concentration of suspended solids was carried into the system from the streets and catch basins. (This is not assumed to be a first flush concept.) The concentration of solids decreased with increasing time intervals from the start of the storm. Since the influent and weir samplers did not activate until the water level in the clarifier reached the level of the weir, the high initial concentration of suspended solids was missed by the influent sampler but collected by the weir sampler.

The manufacturer's operating instructions manual commented that the performance of the tube settlers was a function of many parameters and that the installation of the modules would not automatically improve the performance of the settling basin. Two of the performance parameters were the rate of flow through the modules and the variability of the flow rate. The manufacturer also commented that the tube settlers are most efficient with a constant flow rate not exceeding 4 GPM per square foot of surface area (8 cfs for the Tallmadge Memorial Parkway facility).

The Tallmadge Memorial Parkway retention tank system, however, had varying rates of flow, dependent upon the storm intensity, and generally had flow rates in excess of 223 l/sec (8 cfs). When the actual flow rate was greater than the recommended design flow rate, the efficiency of the tube settlers was not sufficient to prevent the carryover of suspended solids onto the exposed media surface. Three precipitation events which had high flow rates and significantly large concentrations of suspended solids, resulted in a reduction of the infiltration rate into the media. (See recommendations for design modifications to alleviate this condition.) This possibility of a reduction of the media infiltration rate, combined with the rapid runoff from the drainage area and the time involved for personnel to arrive at the site, necessitated the conservative testing procedure which was adopted after the April 1, 1974 storm.

Leaves and other floating debris were not a problem as speculated. During the initial stages of the clarifier filling process, the floating debris which entered the clarifier was transported by the flow toward the effluent end. When the clarifier became 50 percent full, the debris was trapped in a chamber isolated from the tube settlers by the clarifier's transverse structural supports. During the two year operation period, the tube settlers never clogged.

CHLORINATION SYSTEM

Fifteen percent sodium hypochlorite solution was automatically added to the combined sewer flow when the water exceeded the level of the clarifier weir and entered into the tank media. A total of approximately 8350 liters (2200 gal.) of solution was used during the operational period of the prototype facility. The sodium hypochlorite solution was added to retard the BOD requirement of the water and prevent the accumulation of odors and gases during the detention time. During the two-year evaluation period, no significant odors or gases developed. Sodium hypochlorite was not added to those flows which did not exceed the level of the clarifier weir. Many of the precipitation events did not promote flow over the clarifier weir and were drained, after a short detention period, to the Akron sewage treatment plant.

The evaluation of the chlorination system is difficult to access because of the inconclusive sampling and the poor results generated from the colorimetric chlorine analyzers. A small quantity of stormwater from each sample of the weir, tank, and overflow samplers was colorimetrically analyzed for chlorine. However, due to the high turbidity of the stormwater the line feeding the water to the analyzer often clogged and the chlorine readouts which were obtained were inaccurate and inconclusive. Filters were installed in the line prior to the analyzer; however, they did not remedy the situation.

The samples taken from the postchlorination sampling location, above the tube settlers, had average BOD concentrations slightly higher than the prechlorination influent values. Although opposite of that originally anticipated because of the chlorination, these results are understandable. Both the samplers and the chlorinator were activated by the electronic relays in the purge air bubbler (level indicator) only when the water level in the clarifier reached the level of the weir. Therefore, the initial samples collected by the weir sampler were unchlorinated and had slightly higher concentrations of BOD as compared to the influent samples. This was due to any organic material which may have entered into the system at the initiation of a storm as runoff from the drainage area. On seven of the ten events for which BOD data was obtained, the BOD levels of the weir samples were greater than those of the influent samples. Disregarding the data for the April 1, 1974 storm, the average BOD levels obtained by the influent and weir samplers were 33.8 mg/l and 36.5 mg/l respectively. The storm of April 1, 1974 was heavily chlorinated with BOD levels of 119.3 mg/l and 8.2 mg/l for the influent and weir samplers respectively. On this occasion, activation of the samplers was delayed due to a malfunction, therefore, when the weir sampler activated it was collecting chlorinated stormwater. Between the weir and tank samplers there was an average BOD reduction of approximately 30 percent.

CONTAINED SOLIDS WITHIN MEDIA STORAGE AREA

One of the primary objectives of the "Demonstration of Void Space Storage with Treatment and Flow Regulation" was the evaluation of the applicability of inert media, void space storage, "Geo-Cel", for combined sanitary and stormwater flows. The feasibility of the concept was dependent upon the loss of storage volume due to the entrapment of solids within the retention area.

The solids which are encountered in combined sewer systems are classified either as inorganic (sand and silt) or organic matter. The percentages of each are highly variable and are dependent upon the site characteristics, intensity and duration of rainfall, time of year, time interval between storm events, and the time the sample was taken with respect to the initiation of the storm. Dry-weather flows consist of sanitary wastes, the solids of which are primarily organic material. During a precipitation event, however, the storm flow generally reduces the average percentage of organic material and increases the average percentage of inorganic material.

Within the two year operation and evaluation period, twelve selected storm events caused flow to pass over the clarifier weir and into the media storage area. Sampling data was obtained for eleven of the twelve events.

Primarily, equipment malfunctions prevented additional sampling data from being obtained. Based on the sampling data for the eleven storm events, general trends were analyzed to predict the volume of solids which would enter into the media void space for the Tallmadge Memorial Parkway tank. The volume of solids retained in the void space is based on the difference between the solids' mass entering the media during filling of the tank and the solids' mass existing during draining of the tank. With the sampling data that was obtained during the operational period, it was not possible to determine the concentration of suspended solids removed with the effluent from the media.

Table 4 lists all the available significant data for the determination of the volume of solids entering into the media storage area. The values given are averages for each specific storm.

Total solids is defined as the sum of the dissolved and undissolved constituents in the wastewater. Suspended solids are those that float on the surface of, or are in suspension in wastewater, and which are largely removable by laboratory filtering. Volatile solids are those which are lost on ignition of the dry solids at 600°C.

TABLE 4. SOLIDS CONTAINED WITHIN MEDIA

	Weir Sample Averages				Tank Manhole Sample Averages			
Date	# Of	Solids mg/l			# OF	Solids mg/l		
	Samples	Total	SS	Volatile	Samples	Total	SS	Volatile
4-01-74	17	16256	2469	103	-	-	-	-
11-04-74	3	611	259	40*	-	-	-	-
11-20-74	4	378	95	15*	-	-	-	-
1-08-75	12	314	194	45	9	236	71	27
2-23-75	12	332	172	20	10	223	53	7
5-21-75	5	908	842	176	-	-	-	-
5-31-75	12	1066	477	101	12	686	173	56
6-22-75	5	1185	839	105	5	292	144	60
9-05-75	2	551	199	31*	-	-	-	-
9-18-75	3	185	17	3*	3	168	46	14*
10-17-75	7	233	42	6*	8	208	30	9*

*based on an average between volatile and suspended

Generally, the organic solids can be considered as volatile solids. The bulk of the volume associated with organic matter consists of water which naturally comprises about 95 percent of its weight. The values (mg/l) given in Table 4 refer to the dry solids weight. The reduction of retention tank volume, however, was determined by the solids' wet volume. Based on the average concentration of volatile solids in the flow over the weir, an associated wet volume of 1,110 liters of sediment was obtained per million liters of wastewater entering the media. Calculations are presented in Appendix B.

The volume associated with inorganic matter, however, is comprised primarily of solids. The concentration of the inorganic solids (sand and silt) was assumed to be the concentration of the suspended solids minus that of the volatile solids. Based on the average concentration of inorganic solids in the flow over the weir, an associated wet volume of 283 liters of sediment was obtained per million liters of wastewater entering the media. Calculations are also presented in Appendix B.

Comparison of the dissolved solids (total solids minus suspended solids) of the weir samples with those of the manhole samples revealed that only a minor reduction of concentration occurs. Therefore, it was assumed that the dissolved solids will pass through the retention tank with only minor volumetric accumulation within the media.

The total decrease of void space within the media was estimated by the summation of the organic and inorganic volume reductions. Representing the data obtained during the operational period, a total volume reduction of 1393 liters of sediment can be expected per million liters of wastewater passed through the media. This approximation assumes that all the suspended solids which entered the media are trapped within the void space. In reality, however, many of the solids which are contained in the media will exit through the clarifier flap gates upon draining of the retention tank. It was not possible to analyze the concentrations of solids removed from the tank media, and any solids which are present in the media effluent will increase the remaining void space volume. Based on the calculated volume reduction due to solids entering the media, and an average of ten complete fillings per year, over 65 percent of the void space volume would remain after the system life expectancy of 25 years. (The P.V.C. liner has a designed life expectancy of 25 years.)

The leaching of the total solids through the media void space was analyzed by comparing the weir sampler data to that of the tank manhole sampler. Manhole #1, where the sample collection pipe was located, was approximately 30 feet from the exposed media surface. Both the weir samples and the manhole samples are collected by one sampler, sampler #2. Manipulation of the valves for the sampler collection pipes determined which sampling location was being used. During general operation, sampler #2 collected stormwater from the flow over the weir while the media retention tank "Geo-Cel" was being filled. After subsidence of the storm, when flow was no longer passing over the weir, the valves were adjusted to collect samples from manhole #1 of the stored wastewater during drainage of the tank.

Although a substantial time lag between the collection of samples from the two locations was apparent, the data was reasonable in approximating the initial leaching of solids through the media void space. Between the two sampling locations there was an 82 percent average reduction of the suspended solids (refer to Table D-2). The reduction for any specific event may vary from the average and it is anticipated that with continued use of the facility, the leeching effect would become less pronounced due to the filling of the media with sediment.

The reduction of suspended solids, between the samples of the weir and tank manhole #1, was comprised of a 51 percent decrease of the volatile solids and an 86 percent decrease of the inorganic solids. This is a logical distribution since the volatile matter, which is 95 percent water, has a low specific gravity causing it to remain in suspension for longer periods of time as compared to the inorganic matter , which is nearly 100 percent solids and has a higher specific gravity.

Due to the 82 percent decrease in the suspended solids from the weir to manhole #1, it can be predicted that a majority of the solids passing over the weir are trapped by the media near the north wall of the clarifier. A parabolically decreasing concentration of solids retained in the media would be expected with increasing distances from the clarifier. The concentration of solids nearest the clarifier, however, may be reduced during draining of the retention tank since some of the suspended matter will exit the media with the wastewater effluent.

COSTS

The total cost for the experimental demonstration project was $750,000. Of this amount, approximately $471,000 was used for construction of the prototype facility. An itemized breakdown of the construction costs, based on 1972 prices, is given in Table 5.

TABLE 5. CONSTRUCTION COSTS

Description	Est. Quant.	Unit Price	Total
General			
Site preparation	LS	3,000.00	3,000.00
Excavation, backfill and embankments	9300 CY	5.00	46,500.00
Driveway	520 SY	2.50	1,300.00
Rip rap	200 SY	10.00	2,000.00
Effluent pipe bedding	325 CY	13.00	4,225.00
Clarifier	LS	110,937.50	110,937.50
Bottom PVC liner	2894 SY	5.47	15,830.00
Asphalt side liner	1797 SY	3.00	5,391.00
Top PVC liner	4144 SY	2.75	11,396.00

(continued)

TABLE 5. (continued)

Description	Est. Quant.	Unit Price	Total
Geo-Cel media	11,300 CY	7.25	81,925.00
Overflow ditch	550 LF	11.88	6,534.00
Control building	LS	6,535.00	6,535.00
Influent pipe	410 LF	72.00	29,520.00
Overflow pipe	LS	12,500.00	12,500.00
Effluent pipe	465 LF	21.50	9,997.50
Diversion chamber	LS	16,000.00	16,000.00
Manholes	2 Ea	900.00	1,800.00
Instrumentation	LS	2,750.00	2,750.00
Misc. tools	LS	1,200.00	1,200.00
Automatic samplers	3 Ea	5,225.00	15,675.00
Project signs	2 Ea	250.00	500.00
Mechanical			
Heating & ventilating	LS	800.00	800.00
Plumbing	LS	3,000.00	3,000.00
Sodium Hypochlorite sys.	LS	6,000.00	6,000.00
Compressed air sys.	LS	1,000.00	1,000.00
Sampler piping	LS	1,624.00	1,624.00
Plumbing	LS	700.00	700.00
Electrical			
Power & lighting	LS	5,750.00	5,750.00
Instrumentation	LS	34,300.00	34,300.00
Miscellaneous	LS	32,710.00	32,710.00
		TOTAL	471,400.00

Much of the cost which was required in the construction and evaluation of the "Geo-Cel" prototype facility would not be necessary for the installation of a second retention tank. Many of the system components could be simplified thus promoting a reduction of the construction and operating costs. The clarifier could be reduced in size and the tube settlers could be eliminated. The overflow and void space storage feed systems could be redesigned to provide automatic system operation during a storm. Except where restricted by soil conditions, asphalt could be used to completely line the tank. Where site conditions allow, reduced lengths of the influent and effluent pipes and the overflow ditch would be desirable. Other cost reductions could be achieved in future non-research installations by reducing or eliminating the instrumentation and recording equipment. The three observation manholes, control building, and all associated heating, wiring, and plumbing could be eliminated.

The actual cost of future retention tank systems would depend primarily on total volumetric storage capability, type of void space media used, application, and design requirements.

Depending upon the desired use of the facility for combined sewer overflow storage or other uses such as consumer fuels storage, flood control, or industrial chemical and water storage, the design restrictions, inert material used, and resultant costs would vary. Based on the 1972 costs for the Tallmadge Memorial Parkway retention tank, and a combined sewer overflow storage capacity of 3.8 million liters (1 MG.), the total cost of the facility as built was $0.124 per liter of storage. By limiting instrumentation, eliminating the tube settling modules, and redesigning the clarifier section a cost of $0.079 to $0.092 per liter of storage capacity could be expected. Land costs are not included in the figures given. Where any appreciable amount of land must be purchased, the cost per unit of storage would reflect this cost.

Table 6 shows a comparison of the cost of storage capacity for the void space storage concept as compared to underwater storage tanks, underground concrete tanks, and lagoons. The projected cost does not include preliminary studies for each site, extra costs encountered due to specific site conditions, land acquisition, acquisition of all permits, and major alterations to existing structures. Operational costs are also not included.

TABLE 6 CONSTRUCTION COST COMPARISON

Storage Method	Storage Capacity liters	(MG)	Cost ¢/liter	(¢/gal)
Void space storage (Geo-Cel)	3,800,000	(1.0)	7.9-9.2	30-35
Underwater tank	3,800,000	(1.0)	12.7	(48)
Reinforced concrete tank	3,800,000	(1.0)	12.2	(47)
Lagoons	3,800,000	(1.0)	3.5	(13)

Estimated minimum operational costs for the Akron, Ohio prototype facility as constructed were about $3,000 per year. The power requirements for the facility were minimal since filling and draining of the retention tank was not by electrical pumps but by gravity. With minor design changes and minimal funds, the facility could be operated automatically thus drastically reducing the projected yearly operational costs.

AESTHETICS

The underground temporary void space storage concept, "Geo-Cel", with its unique design capabilities incorporating dual land usage, provides a storage facility which is compatible with the surrounding area. This versatility permits adaptation of surface usage to such facilities as recreation, parking, open-air storage, scenic reservation, farming or grazing land, and even the possibility of minor structures.

The nature of the "Geo-Cel" construction offers a safety advantage as a built-in factor. There are no vaults; therefore, no possibility that a gaping surface hole could suddenly form. This is of particular concern where surface dual-usage is a playground, and consideration for the safety of small children paramount.

The Tallmadge Memorial Parkway facility was originally designed to incorporate top surface usage as an adult recreational area. Limited available city funds, however, prevented the grounds from being developed and the area has become an informal trail bike course.

SECTION X

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SECTION XI

PATENTS

The Underground Fluid Storage Tank patent was issued on December 23, 1969, numbered U.S. Patent No. 3,485,049.

The Automatic Sampling Device patent was issued on June 28, 1971 as U.S. Patent No. 3,587,324.

SECTION XII

APPENDIX A

HEAD LOSS THROUGH MEDIA OVERFLOW SYSTEM

When the void space retention area "Geo-Cel" is completely full and stormwater is exiting the 1.22 meter (48 in.) overflow risers, the overflow flow lines are assumed to resemble those presented in Figure A-1. The accuracy of the calculations are dependent upon the "in-situ" characteristics of the media and the precision of the flow analysis. Since the media specifications are vague, an in depth flow analysis would be unwarranted. The overflow system was divided into five approximate flow channels (Figure A-1) and these channels evaluated for their maximum potential flow rate.

The flow through a porous media can be determined by Darcy's equation:

$$Q = kiA \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A-1)}$$

where Q is the flow rate through the media, k is the permeability of the media, i is the hydraulic gradient, and A is the cross-sectional flow area.

The permeability of the porous media is a significant factor in the determination of the rate of flow through the media. Permeability is highly variable and dependent upon many media characteristics, however, as an approximation, Cedergren states that permeability is a function of the "effective" diameter of the pore spaces in the media. The "effective" pore diameter is:

$$d_{eff} \approx 1/5\ (D_{10}) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A-2)}$$

where D_{10} is the stone diameter of which 10 percent is finer by weight.

According to specifications, the bulk of the aggregate placed within the retention tank is AASHO #1 or #2 size with 10 to 15 cm (4 to 6 in.) of AASHO #46 size covering the exposed media surface.

The bulk of the media, therefore, has an "effective" pore diameter of approximately 7.62 mm (0.3 in.). The cover stone

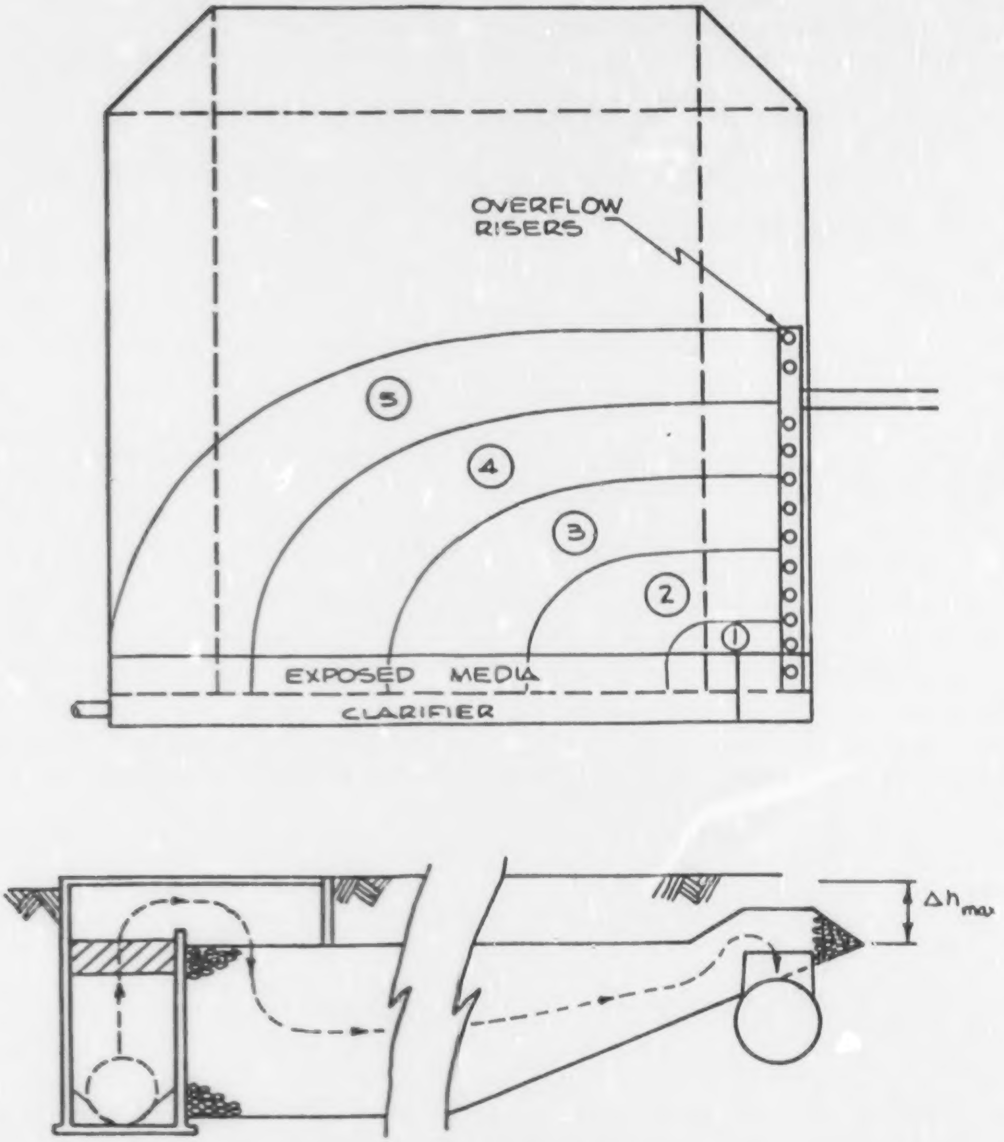

FIGURE A-1. OVERFLOW SYSTEM ANALYSIS

has an "effective" pore diameter of approximately 1.91 mm (0.075 in.). Cedergren also presented a chart, Figure A-2, which correlates the "effective" diameter with the velocity of flow given that the hydraulic gradient, i, equals 0.01.

From Figure A-2 we obtain:

V_{cover} = 0.0027 m/sec (0.009 fps) for d_{eff} equal to 691 mm (0.075 in.)

and

V_{bulk} = 0.0122 m/sec (0.04 fps) for d_{eff} equal to 7.62 mm (0.03 in.)

However, since the hydraulic gradient was 0.01, the permeabilities of the two aggregates are:

k_{cover} = 0.27 m/sec (0.90 fps).................(A-3)

and

k_{bulk} = 1.22 m/sec (4.0 fps)...................(A-4)

The head loss through two permeabilities can be calculated on the basis that the rate of flow which passes through the cover aggregate will also pass through the rest of the media:

$Q_1 = Q_2$..(A-5)

where the subscripts 1 and 2 refer to cover the bulk respectively. Based on equation (A-1) and rearranging the variables:

$$\Delta h_1 = \left(\frac{k_2\ A_2\ \Delta L_1}{k_1\ A_1\ \Delta L_2}\right)\Delta h_2 \text{ ....(A-6)}$$

The maximum allowable headwater is equal to the elevation of the bottom of the cover tee's minus the elevation of the rim of the 1.21 meter (48 inch) risers. Therefore:

$\Delta h_1 = \Delta h_2$ = 0.96 m (3.15 ft).....................(A-7)

The flow lengths, ΔL, can be determined by analyzing Figure A-1. ΔL_1 is equal to the thickness of the cover material. ΔL_2 is equal to the average flow length for each of the flow channels involved:

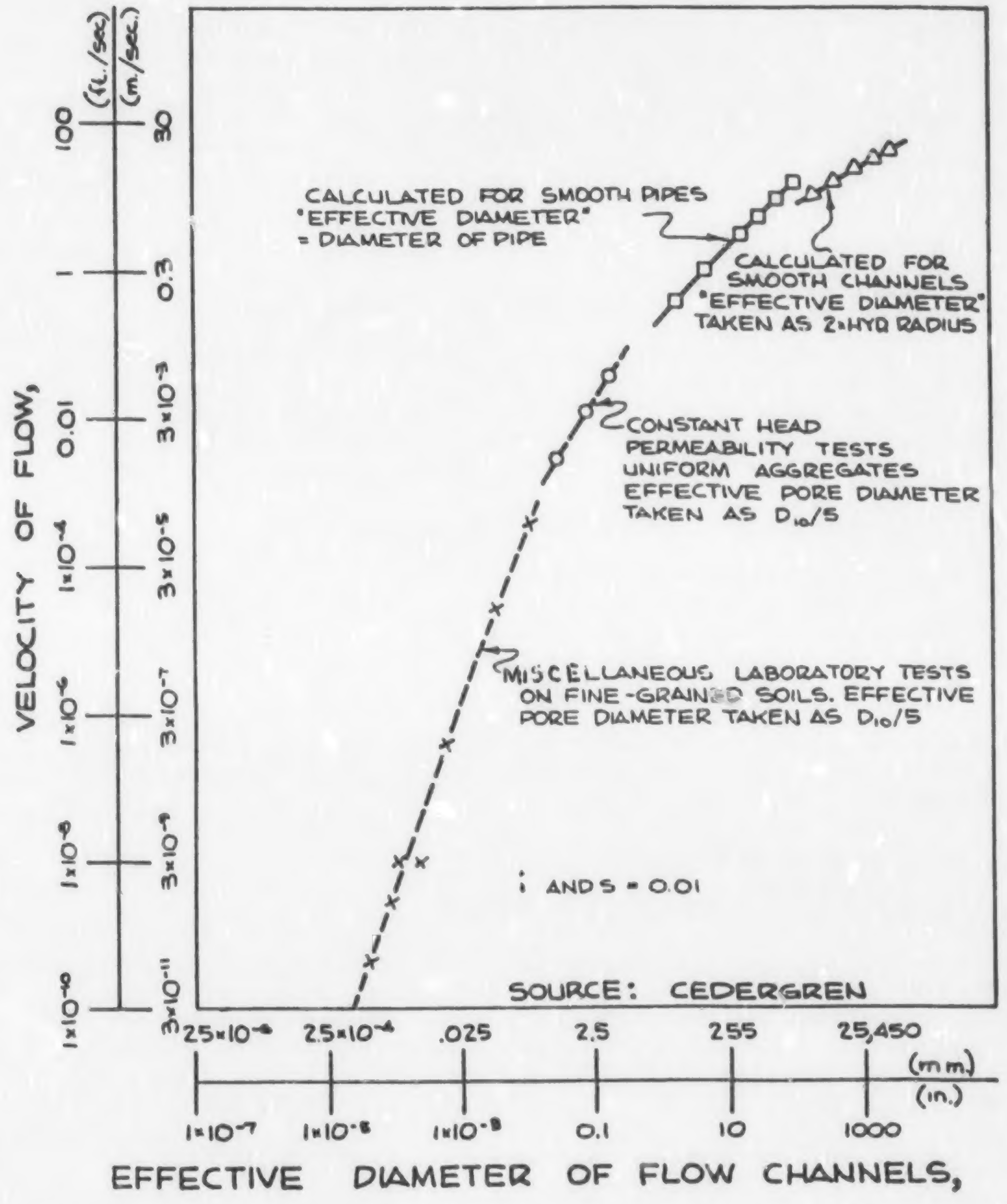

FIGURE A-2. VELOCITY OF FLOW VS. EFFECTIVE DIAMETER

$\Delta L_1 \simeq 0.15$ m (0.5 ft)

$\Delta L_{2-1} \simeq 9$ m (30 ft) w/ $A_{2-1} \simeq 14$ m^2 (150 ft^2)
$\Delta L_{2-2} \simeq 21$ m (70 ft) $A_{2-2} \simeq 22$ m^2 (240 ft^2)
$\Delta L_{2-3} \simeq 38$ m (125 ft) $A_{2-3} \simeq 28$ m^2 (300 ft^2)
$\Delta L_{2-4} \simeq 55$ m (180 ft) $A_{2-4} \simeq 28$ m^2 (300 ft^2)
$\Delta L_{2-5} \simeq 70$ m (230 ft) $A_{2-5} \simeq 28$ m^2 (300 ft^2)

Substitution into equation (A-6) gives:

$$\Delta h_1 = .022(A_2/\Delta L_2) \qquad (0.06-\Delta h_1) \text{.........(A-8)}$$

Trial and error solution of equation A-8 yields:

$\Delta h_{1-1} = .03$ m (.10 ft) $\Delta h_{2-1} = .93$ m (3.05 ft)
$\Delta h_{1-2} = .02$ m (.07 ft) $\Delta h_{2-2} = .94$ m (3.08 ft)
$\Delta h_{1-3} = .015$m (.05 ft) $\Delta h_{2-3} = .945$ m (3.10 ft)
$\Delta h_{1-4} = .01$ m (.03 ft) $\Delta h_{2-4} = .95$ m (3.12 ft)
$\Delta h_{1-5} = .01$ m (.03 ft) $\Delta h_{2-5} = .95$ m (3.12 ft)

Substitution back into equation A-1 gives:

$Q_1 = 1730$ l/sec (61 cfs)
$Q_2 = 1190$ l/sec (42 cfs)
$Q_3 = 850$ l/sec (30 cfs)
$Q_4 = 590$ l/sec (21 cfs)
$Q_5 = 450$ l/sec (16 cfs)

The total maximum overflow rate is equal to the sum of the individual channel flows therefore at maximum allowable headwater:

$Q_{\text{overflow}} \simeq 4810$ l/sec (170 cfs)

INFILTRATION RATE INTO EXPOSED MEDIA

When the void space retention area "Geo-Cel" is only partially full the infiltration rate, or filling rate, is controlled by the permeability of the exposed media. According to the specifications the exposed media consists of AASHO #46 size stone. The permeability of the stone is approximately 0.27 m/sec (0.90 fps). An infiltration rate, calculated by equation A-1, of nearly 42,500 l/sec (1500 cfs) could be obtained without any pooling of stormwater on the exposed media surface.

The media, however, can trap the suspended solids that are carried over the clarifier weir. If a significant quantity of solids would be retained on the exposed media surface, the infiltration rate into the media would be reduced. On April 1, 1974 a severe storm, 75 year recurrence interval, deposited fine sand on the exposed media surface. Assuming that a fine silty-sand, with a permeability of 0.0001 cm/sec (3.3×10^{-6} fps), was deposited at an average depth of 5 cm (2 in.), the infiltration rate into the media would be drastically reduced. Based on a maximum allowable headwater of 0.96m(3.15 ft.), the infiltration rate into the media is approximately 3 l/sec (0.1 cfs).

Since the allowable infiltration rate (0.1 cfs) was considerably less than the influent flow rate, three cover sections were lifted slightly out of place by the forces developed within the clarifier. The clarifier cover tees were designed to alleviate the pressures developed during extreme flow conditions. The tee sections were easily moved into place after the April 1st storm repairs were completed.

APPENDIX B

REDUCTION OF VOID SPACE VOLUME

Based on the weir sampling data obtained for eleven precipitation events (Table 7), the reduction of void space volume can be estimated assuming that the suspended solids which flow over the clarifier weir are trapped within the media. The suspended solids are composed of both organic and inorganic solids. The average concentration of organic solids is assumed to equal the average concentration of volatile solids. The average concentration of inorganic solids is equal to the average of the suspended minus the volatile solids.

Avg. organic concentration	59 mg/l
Avg. inorganic concentration	451 mg/l

Organic sludge is comprised primarily of water, 95 percent, with a specific gravity approximately equal to 1.06. The volume reduction due to organic solids is based on the wet weight, therefore:

Organic Volume	59 ppm x 8.34 lb/gal x (100/5)x[7.48 gal/cf/(1.06x62.4 lb/cf)]
Organic Volume	1110 liters wet organics per million liters of stormwater

Inorganic material is comprised primarily of solids (estimated at 75 percent) with a specific gravity approximately equal to 2.125. The volume reduction due to inorganic solids is also based on the wet weight, therefore:

Inorganic Volume	451 ppm x 8.34 lb/gal x (100/75)x[7.48 gal/cf/(2.125x62.4 lb/cf)]
Inorganic Volume	283 liters wet inorganics per million liters of stormwater

APPENDIX C

OPERATIONS MANUAL

An operations manual for the pilot facility in Akron, Ohio was prepared and submitted to the City of Akron Sewer Maintenance Division upon turnover of the facility at the completion of the two year operational period. The operations manual provided a detailed description of the system components and design and their function in the operation of the facility. Some of the major items included were:

1. Analysis of the drainage area and anticipated runoff,
2. Flow regulation and control devices,
    a. Diversion manhole baffle wall and orifice
    b. Effluent slide gate and theoretical flow rates
    c. Overflow slide gate and theoretical flow rates
3. Detailed description of the control building instrumentation, their operation, maintenance, test checks, and adjustments,
4. Facility storage operations,
    a. Prior to a storm
    b. During a storm
    c. After a storm

Selected portions of items 3 and 4 of the operations manual are presented to familiarize the reader with some of the precision equipment which was incorporated in the prototype facility.

Annunciator Lights

The annunciator panel lights located at the top of the control cabinet (Figure C-1) provide a visual alarm indication of significant conditions. There are 7 messages. When activated the appropriate panel light will blink until the acknowledge button is pushed. The "ACK" button is on the right side adjacent to the lights. If the condition is true, the appropriate light will remain lit when acknowledged, if the condition is false or has already passed, the light will go out.

Combustible Gas Alarm --
This light will activate if there is an accumulation of flammable gases in the clarifier section. Sewage can potentially give off methane. To date, over the two year operation period, there have been no such accumulations. One of the monitor detection

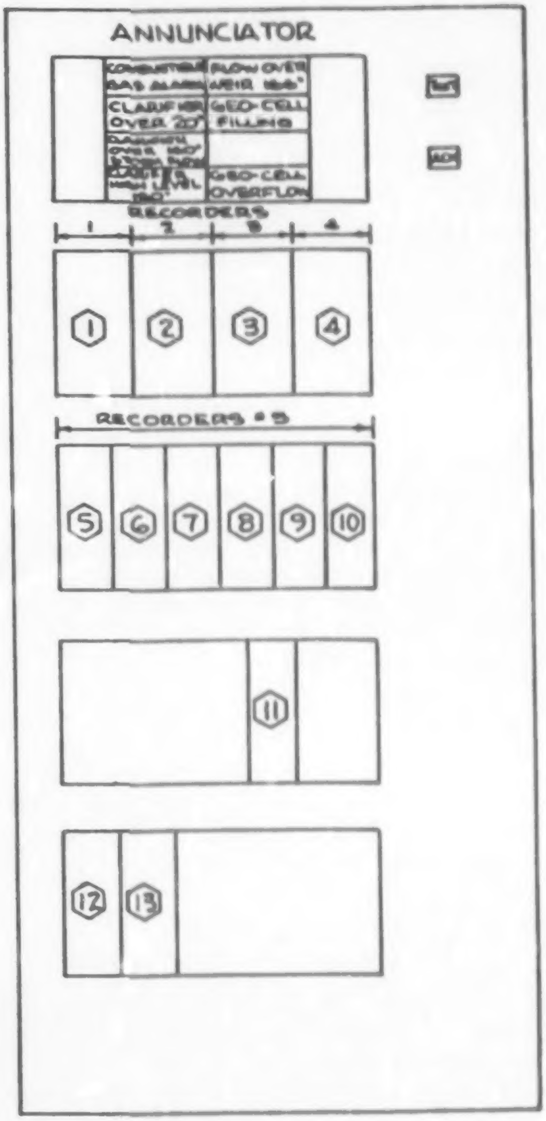

FIGURE C-1. ANNUNCIATOR PANEL LIGHTS AND RECORDERS head devices is located under the control building and access is via the external hatchway. If activated, the clarifier gas monitor will trigger the annunciator panel light "Combustible Gas Alarm" . A second detection head is located at the base of the plywood instrument mounting board. This monitor covers the region inside the control building. Should combustible gas be detected in the control building a loud audible horn alarm will sound and a red globe warning light, mounted outside the control building over the entrance doorway, will activate. In the event of a positive alarm condition personnel should notify their supervisors, turn off unnecessary electrical devices to eliminate sources of ignition, open all hatchways, and provide ventilation for dilution of concentrations of hazardous gases.

Clarifier Over 20" --
This signal light provides the first indication that the water level in the clarifier is rising, and is above the top of the effluent slide gate.

Clarifier Over 160" Storm Flow --
This signal light indicates the level is almost to the point of flow over the weir. The chlorine pump should automatically energize at this time. The chlorine pump is located in a pump pit at the influent end of the clarifier.

Flow Over Weir 166" --
This signal light indicates that the clarifier is full and stormwater should flow over the weir elevation and begin to fill the tank. The chlorine ratio controller located on the lower right side of the control cabinet should activate at this time indicated by deflection of the green needle pointer in the ratio controller. This annunciator light (and ratio controller) is activated by a relay from the air pressure bubbler located beneath the control building hatchway. The air bubbler gives a pressure reading produced by the clarifier water level.

Geo-Cel Overflow --
This device operates via a pressure diaphram located in the overflow pipe. This signal light is activated when the level of flow in the pipe exceeds 7.6 centimeters (3 in.) which indicates that either the tank has reached capacity, or the 1.52 meter (60 in.) gate is open to bypass.

Clarifier High Level 180" --
This signal light indicates that the freeboard above the weir is being exceeded. Flow over the weir is at 4.22 meters (166 in.), at 4.78 meters (188 in.) flow will exceed the clarifier tee sections. The slide gates should be adjusted to prevent a high level overflow condition.

Geo-Cel Filling --
This signal light activates when the water level in the storage tank exceeds 7.6 centimeters (3 in.) in depth. The sensor, located in depth manhole #1 nearest the control building, cannot respond to water levels less than 7.6 centimeters. Geo-Cel filling will occur when there is flow over the clarifier weir or infiltration through the flap gates.

Monitoring Devices

Also located on the annunciator panel are a number of items (ten chart recorders, one chlorine ratio controller, and two combustible gas monitors) which monitor and record the flow through the prototype facility.

Function #1a: Rain Gauge --
A Belfort rainfall gauge is located in the control building behind the annunciator panel. The collection funnel is located on the roof with a rubber hose transferring the rainfall. The gauge was calibrated to a 25.4 centimeter (10 in.) full scale to provide a direct chart scale reading. A set of weights is provided to check the device. The chart is changed at the end of each month and the collection bucket emptied. The rain chart records in red ink. A potentiometer is mounted on the concrete wall by the rain gauge which is used to zero and span the chart recorder.

Function #1b: Ground Water Level Monitor --
The blue pen of the two pen chart recorder indicates the ground water level in relation to the Geo-Cel tank. There is a bubble level device inside a 10 centimeter (4 in.) well casing located 6 meters (20 ft.) NE of the control building which measures ground water level. Typically the dry-weather ground water level is 5 percent of full scale on the recorder chart, rises to 10 percent within twenty minutes after a storm, and falls slowly over several hours. This recorder measures a depth of 0 to 5.1 meters (200 in.) of ground water with respect to the bottom elevation of 240.8 meters (790.0 ft). The ground water level also rises in relation to spring thaw conditions. A sharp rise during a tank filling or emptying operation may indicate a rupture in the tank membrane liner.

Function #2: Clarifier Level --
The level of combined sewage flow in the rectangular clarifier section is recorded on a two pen chart recorder in the annunciator cabinet. The red pen records from 0 to 51 centimeters (20 in.) of flow. When twenty inches is exceeded a light "Clarifier Over 20 inches" in the annunciator alarm panel will begin to blink. The blue pen measures water level from 0 to 4.78 meters (188 in.). When the water level in the clarifier reaches 4.22 meters (166 in.), water should flow over the weir and into the tank. The remaining 0.56 meters (22 in.) between

the top of weir and the bottom of the concrete tees is freeboard. If it appears that the clarifier level of 4.78 meters will be reached the slide gate valves should be manipulated to relieve the excess flows. This will normally occur only when flows received exceed the acceptance rate of the Geo-Cel media. This condition can be caused by fine particles or debris which may reduce the infiltration rate into the media (Appendix A). Table C-1 lists significant clarifier levels.

TABLE C-1. SIGNIFICANT CLARIFIER LEVELS

Item	Meters	Inches	% Full
20 inch alarm light	0.51	20	11
Bottom of flap gates	0.69	27	14
Alarm probe	0.61 to 1.22	24 to 48	12 to 25
Bottom of 1.52 meter (60 in.) bypass	1.85	73	39
Top of 1.52 meter (60 in.) gate opening	2.41	95	51
Weir	4.22	166	88
High Water Level	4.57	180	96
Maximum Water Level	4.78	188	100%

Function #3 --
This records the clarifier level between 4.22 meters, the point of flow over the weir, to 4.78 meters, the point of overflow of the concrete clarifier tees.

Function #4 --
This records the flow exiting the 1.52 meter bypass pipe for depths between 7.6 centimeters (3 in.) to 1.52 meters (60 in.) (The sensor cannot respond to levels less than 7.6 centimeters). The red pen charts the depth on a scale from 0 to 100 percent.

Function #5 --
This records water depth in the tank manhole nearest the control building from 0 to 3.05 meters (0 to 100%) with a bottom elevation of 242.9 meters (797.1 ft.).

Function #6 --
This records water level in the middle tank manhole from 0 to 3.05 meters (0 to 100%) with a bottom elevation of 243.0 meters (797.34 ft.)

Function #7 --
This records water level in the northwest tank manhole from 0 to 3.05 meters (0 to 100%) with a bottom elevation of 243.3 meters (798.23 ft.)

Function #8 --
This records the chlorine residual in the water flowing over the weir as indicated by the Hach Chlorine Residual Analyzer, 0 to 5 PPM full scale.

Function #9 --
This records the chlorine residual in the overflow effluent should the Geo-Cel overflow.

Function #10 --
This is an extra blank channel. Should one of the recorders in this section malfunction, its function could be transferred to this station.

Function #11: Chlorine Ratio Control Station --
This device operates via the bubbler air pressure to control the valve regulating the position of the chlorine feed valve from a .3 to 3.0 ratio. The setting can be changed by adjusting the scale pointer slide. The chlorine is added as a 15 percent sodium hypochlorite solution by a 190 liter per minute (50 GPM) pump.

Function #12: Clarifier Combustible Gas Monitor --
This indicates that the combustible gas in the clarifier has reached 40 percent of the lower explosive level. This will trigger both the alarm light in the annunciator panel and an audible horn. The meter is located on the lower left side of the control panel.

Function #13: Control Building Combustible Gas Monitor --
This indicates percentage of lower explosive level of combustible gas in the control building. If activated this will trigger the external red globe over the entrance doorway. This is located on the lower right of the control cabinet.

Mounting Board Instrumentation

Adjacent to the control panel is the instrument board which houses the interface devices that convert the pneumatic pressure signals to electrical signals. Figure C-2 shows the instrument mounting board and the locations of the various controls.

A items - are electric-pressure transmitters for the depth recording devices: A1 is for the overflow pipe; A2, A7, and A8 are for the three tank depth monitors; A3, A4, and A5 are for the clarifier level monitor; and A6 is for the water table monitor.

B items - are the filter type pressure regulators.

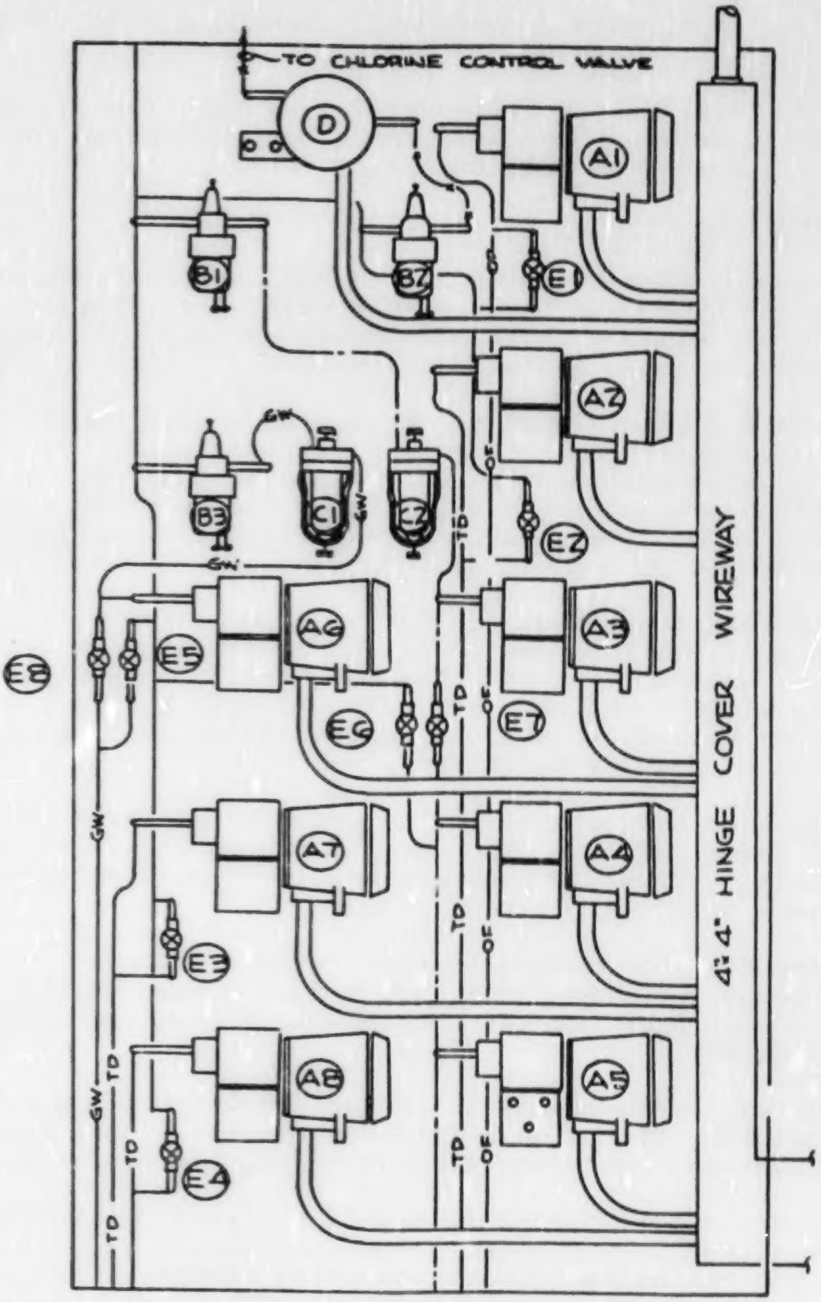

FIGURE C-2. INSTRUMENT MOUNTING BOARD

C items - are the site feed bubblers. These are for the visual confirmation of air pressure in the lines.

D items - is an electro-pneumatic transducer for the chlorine control valve.

E items - are high pressure air valves for blowing out the air lines and performing test checks on the recording instruments.

Instrumentation Test Checks

The annunciator panel warning lights and chart ink pen response can be tested by regulating the appropriate air pressure knobs located on the plywood instrument mounting board (Figure C-2). Each blowdown control has a function:

E1 triggers a test signal to the pressure diaphram in the overflow pipe. Upon triggering a signal, the "Geo-Cel Overflow" annunciator panel light should blink. The overflow sampler motor will start, the overflow chlorine analyzer needle will deflect and the tank overflow recorder #4 pen should also deflect.

E2 triggers a test signal to the pressure diaphram located in tank depth well #1 (nearest control building). The test signal should cause the "Geo-Cel Filling" annunciator panel light to blink. The weir chlorine analyzer needle and the tank depth well #1 pen (recorder #5, channel #1, should respond by deflecting.

E3 triggers a test signal to the pressure diaphram located in tank depth well #3. The test signal should initiate a pen deflection from the tank depth well #3 pen (recorder #5, channel #3).

E4 triggers a test signal to the pressure diaphram located in tank depth well #2. The test signal should initiate a pen deflection from the tank depth well #2 pen (recorder #5, channel #2).

E5 triggers a test signal to the ground water table bubbler. (Close E8) This signal should cause the ground water table recorder pen (recorder #1, pen #2 (blue)) to deflect.

E6 triggers a test signal to the clarifier level bubbler. Regulation of this blowdown control should trigger (close E7) the following annunciator panel lights: Clarifier over 20", Clarifier over 160", Clarifier over 180", and Clarifier over 166". The Clarifier 0-20", Clarifier 0-188" and Clarifier 166-188" recorder pens (recorder #2 pen #1, recorder #2 pen #2, and recorder #3 respectively) should respond by deflecting. The signal should also activate the relay in the chlorine fuse box; an audible click can be heard.

E7 is an isolation valve which protects the clarifier bubbler and regulator. Before opening the E6 blowdown control knob, the E7 knob must be closed. Otherwise E7 should remain open.

E8 is an isolation valve which protects the ground water bubbler and regulator. Before opening the E5 blowdown control knob, the E8 knob must be closed. Otherwise E8 should remain open.

Required Maintenance

1. Drain compressor of condensed moisture by opening the bottom drain. The manufacturer's manual suggests doing this once per week.
2. Test the clarifier water probe monthly, by immersing in water and checking with the Sewer Maintenance Division to determine if the alarm light functions. The copper prongs of the probe should be cleaned with emery cloth to a bright surface once a month.
3. Change rain gauge, clarifier depth and tank depth charts once per month.
4. Change ink cartridges on the rain gauge and clarifier level every four months.
5. Check annunciator panel lights by pressing "Test" button. Press the "Acknowledge" button to stop.
6. Check the various control functions by individually opening (and then closing) the air pressure valves located on the plywood panel.
7. Grease the gears on the slide gate pedestals.
8. Oil various door and hatch hinges as needed.
9. Verify setting of chlorine ratio controller and auto button on samplers.
10. Inspect diversion manhole monthly for integrity of the orifice. The orifice limits the stormflow to the clarifier.
11. Check fuse box and chlorine fuse box. This is located on the south wall adjacent to the plywood instrument mounting board.

12. Check calibration of rain gauge with weights provided. Empty bucket at end of month and reset. This is explained under the section on test checks and adjustments.

13. Check remaining amount of chlorine monthly, by opening the underground holding tank bolted coverlid and measuring manually with a dip rod. The chlorine is in the form of a 15 percent liquid sodium hypochlorite solution.

    Caution: Chlorine is caustic to the skin and eyes. The tank holds a maximum of 5700 liters (1500 gal.). A 5.1 centimeter (2 in.) filler pipe is located adjacent to the clarifier south wall approximately 7.6 meters (25 ft.) east of the chlorine tank.

Operations Prior To A Storm

1. Preadjust and check periodically the influent pipe orifice opening within the diversion manhole to obtain a maximum desirable flow rate. Upon turnover of the system to the City of Akron, the orifice opening was 25.4 centimeters (10 in.) which allows a maximum influent flow rate of approximately 990 liters per second (Figure 12). The diversion wall height is equal to 1.22 meters (4 ft.)

2. The probe alarm, which signals upon filling of the clarifier, can be adjusted to any height. The probe, however, should be set at a height greater than 0.6 meters (2 ft.) to reduce triggering of the signal by slight increases of the dry-weather flow, yet less than 1.22 meters (4 ft.) to allow a maximum personnel response time.

3. The 46 and 152 centimeter (18 and 60 in.) gate valves control the flow exiting from the clarifier and can be adjusted in anticipation of a storm. The 46 centimeter gate should remain at least 10 centimeters (40 turns) high to allow dry-weather flow, up to 71 liters per second (2.5 cfs), to pass through the clarifier without triggering the probe alarm. The 152 centimeter gate can be regulated between 0 and 56 centimeters high, allowing up to 2550 liters per second (90 cfs) to exit from the system. If the orifice opening within the diversion manhole is less than 51 centimeters (20 in.), and the 1.52 meter gate is open 56 centimeters then no flow will pass over the clarifier weir and into the storage media. This flow, however, is discharged directly into the river and defeats the purpose of the retention tank. When the 1.52 meter gate is closed, the clarifier filling rate is dependent upon the intensity of the rainfall; however, the allowable response time for personnel to get to the site is short. Personnel should be on the site for observation

and regulation of the storm flow. There have been instances in the past (4-1-74) where the infiltration rate into the exposed media was reduced by sand and silt being deposited on the media surface.

4. Check functioning of various devices (clarifier charts, samplers, etc.) by using the blowdown controls. This is explained under the section on test checks and adjustments.

Operations During A Storm

1. When the probe alarm signals, personnel should be dispatched to the retention tank site to monitor and regulate the 0.46 meter (18 in.) and 1.52 meter (60 in.) slide gates.

2. If the 1.52 meter gate valve is open, it should be closed to prevent any overflow into the river. The 0.46 meter gate can remain at the dry-weather flow height.

3. Careful observation of the clarifier filling chart is necessary to assure that flow is infiltrating into the media. At 87 percent, flow is passing over the weir and into the media. If the clarifier level exceeds 90 percent, check the retention tank depth recordings. If the retention tank is not full, the water is not infiltrating properly. Visual observation through the hatch gate outside the building should be made to check pooling on the exposed media surface. If pooling on the media is occurring, the 1.52 meter gate should be opened immediately to prevent excessive pressure buildup within the clarifier.

4. Sampling: Check to make sure the samplers activate at the proper times, otherwise regulate manually. Label samples according to time and location. Take grab samples as needed. Reload sample jugs and change valving to take samples from tank as it drains. Regulate valve opening to get bottle filled in the required amount of time.

5. Chlorination: Observe the Hach Chlorine Analyzer for priming and operation: Adjust the chlorine ratio control as needed.

Operations After A Storm

1. After the storm has subsided, the retention tank and clarifier should be drained through the 0.46 meter effluent gate opening. The 1.52 meter gate should remain closed to prevent discharging storm flow directly into the river.

2. The 0.46 and 1.52 meter gate valves should be adjusted to the desired dry-weather flow openings.

3. The clarifier filling actuator probe should be cleaned with emery cloth or steel wool and adjusted to the proper elevation. This should be done after each storm.

4. The sample bottles should be replaced. The sampler valve should be reset to take the next sample from the clarifier. The check valve on the pumps should be opened and closed to drain residual stormwater in the pickup lines. The filling nozzle should be reset at the beginning bottle.

5. Turn off potable water, heat, lights and secure door.

The operations which were presented are those that were required for the prototype facility. The City of Akron, however, has planned for minor alterations to the prototype facility to provide for a more automatic system operation with less data evaluation.

APPENDIX D

TABLE D-1. PRECIPITATION EVENTS

Date	i (cm/hr)	t (min)	Q* (1/sec)	Gate Openings (m) Orifice	Overflow	Effluent	Water Level (%) Clar.	Tank	Comments
1-19-74	0.19	840	74	1.22	0	0.15	10		
1-23-74		210		1.22	0	0	70		
1-27-74	0.61	25	102	1.22	0	0.15	17		
1-28-74	0.38	60	85	1.22	0	0.15	15		
2-22-74	0.11	60	99	1.22	0	0.10	23		DWF = 3-1/2"
2-28-74	1.02	15	227	1.22	0	0.10	47		
3-04-74	0.51	30	113	1.22	0	0.10	33		
3-05-74	0.25	30	76	1.22	0	0.10	25		
3-07-74	0.51	30	122	1.22	0	0.10	17		
3-08-74	0.34	45	57	1.22	0	0.10	17		
3-08-74	1.52	35	578	1.22	0	0.10	90		lost baffles
3-09-74	3.81	20	173	1.22	0	0.10	39		lost baffles
3-19-74	0.25	60	91	1.22	0	0.10	22		
3-19-74				1.22	0	0			DWF test
3-29-74				1.22	0				data lost
4-01-74	3.05	5	159	1.22	0	0	65		
4-01-74	4.32	60	640	1.22	0→0.56	0.10→0.46	100	20	media clogged
4-03-74	3.56	15	722	1.22	0.56	0.46	50		
4-03-74	1.52	10	612	1.22	0.56	0.46	52		
4-04-74	1.02	15	800	1.22	0.56	0.46	55		
4-14-74	2.67	20	2830	1.22	0.56	0.46	74		
4-22-74	0.34	45	238	1.22	0.56	0.46	10		
4-30-74	0.34	270	283	1.22	0.56	0.46	10		gate open
4-30-74	3.05	20	1105	1.22	0.56	0.46	63		
5-06-74				1.22					baffles removed
5-09-74	2.29	10		1.22	0.56	0.46			until repairs
5-09-74	1.02	15		1.22	0.56	0.46			completed

DWF: dry weather flow

*Avg. clarifier filling rate

(continued)

TABLE D-1. (continued)

Date	i (cm/hr)	t (min)	Q* (l/sec)	Gate Openings (m)			Water Level (%)		Comments
				Orifice	Overflow	Effluent	Clar.	Tank	
5-11-74	1.20	70		1.22	0.56	0.46	6		
5-22-74	0.95	40		1.22	0.56	0.46	5		
5-22-74	1.52	10		1.22	0.56	0.46	3		
5-23-74	0.85	45		1.22	0.56	0.46	3		
5-24-74	0.34	45		1.22	0.56	0.46	2		
5-29-74	1.03	170		1.22	0.56	0.46	5		
6-15-74	0.51	30		1.22	0.56	0.46	1		
6-15-74	0.44	35		1.22	0.56	0.46	1		
6-19-74	3.81	10		1.22	0.56	0.46	5		
6-24-74				1.22	0.56	0.46			lost data
6-26-74				1.22	0.56	0.46			lost data
6-30-74				1.22	0.56	0.46			lost data
8-02-74	3.56	15		0.25	0.56	0.46	20		Baffles replaced
8-08-74	5.72	12		0.25	0.56	0.46	27		w/7" opening
8-08-74	1.37	20		0.25	0.56	0.46	3		at bottom
8-08-74	2.74	10		0.25	0.56	0.46	2		7-23-74
8-09-74	0.27	45		0.25	0.56	0.46	3		and orifice
8-11-74	1.37	20		0.25	0.56	0.46	6		installed
8-11-74	3.05	5		0.25	0.56	0.46	2		
8-12-74	5.33	10		0.25	0.56	0.46	21		
8-13-74	6.33	13		0.25	0.56	0.46	50		
8-13-74	0.25	60		0.25	0.56	0.46	1		
8-17-74				0.25	0.56	0.46	45		rain data lost
8-28-74	0.61	50		0.25	0.56	0.46			clarifier
8-28-74	0.34	45		0.25	0.56	0.46			chart data
8-29-74	0.51	90		0.25	0.56	0.46			lost
8-29-74	1.35	45		0.25	0.56	0.46			
8-31-74	4.76	8		0.25	0.56	0.46	50		
9-03-74	0.25	60		0.25	0.56	0.46			
9-12-74	3.05	5		0.25	0.56	0.46	15		

*Avg. clarifier filling rate

(continued)

TABLE D-1. (continued)

Date	i (cm/hr)	t (min)	Q* (1/sec)	Gate Openings (m) Orifice	Overflow	Effluent	Water Level (%) Clar.	Tank	Comments
9-13-74	2.86	8		0.25	0.56	0.46	5		
9-17-74	6.10	15		0.25	0.56	0.46	50		
9-27-74	3.05	10		0.25	0.56	0.46	45		
9-28-74	1.02	15		0.25	0.56	0.46	2		Replaced bottom
10-07-74				0.25					baffle boards
10-14-74				0.25	0	0.10	20		test
10-25-74	1.52	10	280	0.25	0.56	0.10	47		
11-03-74	3.81	20	368	0.25	0.56	0.10	53		No notification
11-04-74	0.14	480	59	0.25	0	0.10	87	20	test
11-20-74	2.03	30	1048	0.25	0.56	0.10	53		
11-20-74	0.25	120	1104	0.25	0.56→0	0.10→0	90	20	
12-05-74				0.25	0.56	0.10			compressor mal-
12-24-74	0.34	45	110	0.25	0	0.10→0.05	35		function
12-31-74				0.25	0	0.05	80		test
1-08-75	0.51	150	255	0.25	0	0.05	89	64	
1-08-75	0.38	40	142	0.25	0	0.46	11		
1-25-75	0.34	45	113	0.25	0	0.11	76		
1-29-75	0.38	60	105	0.25	0	0.11	35		
1-29-75	3.05	5	408	0.25	0	0.11	57		
2-07-75				0.25	0	0.06			
2-12-75	0.25	60		0.25	0	0.06			snow
2-17-75	0.25	60		0.25	0	0.06	9		
2-22-75	0.25	60	57	0.25	0	0.06	8		
2-23-75	0.25	60	57	0.25	0	0.06	10		
2-23-75	0.36	165	277	0.25	0	0	87		
2-23-75	1.02	60	405	0.25	0	0	87		
3-07-75	0.68	45	198	0.25	0	0.10	74		
3-12-75	0.25	60	76	0.25	0	0.10	22		
3-19-75	0.25	60	62	0.25	0	0.10	13		power off 3-18-75
3-24-75	0.51	45	224	0.25	0	0.10	55		

*Avg. clarifier filling rate
(continued)

TABLE D-1. (continued)

Date	i (cm/hr)	t (min)	Q* (l/sec)	Gate Openings (m) Orifice	 Overflow	 Effluent	Water Level (%) Clar.	 Tank	Comments
3-24-75	0.51	14		0.25	0	0.10	30		
4-19-75	0.25	60		0.25	0	0.10	10		
4-19-75	1.52	5		0.25	0	0.10	15		
4-20-75	0.34	45	207	0.25	0	0.10	45		
4-24-75				0.25	0	0.10	23		
4-27-75				0.25	0	0.10	27		thaw
4-28-75				0.25	0	0.10	21		
4-30-75	0.25	60	113	0.25	0	0.10	40		late notification
5-03-75	1.52	5	99	0.25	0	0.10	35		
5-04-75	0.34	45	178	0.25	0	0.10	59		
5-06-75	0.34	45	176	0.25	0	0.10→0	48		
5-06-75	0.25	60		0.25	0	0	48		
5-20-75	0.51	75	340	0.25	0	0.10	87		
5-21-75	4.57	40		0.25			87	20	pooling on media
5-21-75	3.05	5		0.25	0.56	0.46	22		
5-21-75	2.29	20	425	0.25	0.56	0.46	27		
5-21-75	2.29	20	396	0.25	0.56	0.46	22		
5-22-75	1.52	10	368	0.25	0.56	0.46	20		
5-23-75	1.02	15	368	0.25	0.56	0.46	20		
5-25-75	2.03	30	481	0.25	0.56	0.46	28		
5-27-75				0.34	0.56	0.36			orifice at 13-1/2"
5-29-75	1.02	15	396	0.34	0	0.10	79		sandbagged
5-29-75	3.05	5	85	0.34	0	0.10	19		baffle
5-29-75	1.52	5		0.34	0	0.10	41		
5-31-75	2.03	60	510	0.34	0	0.15→0	90	58	pooling on media
5-31-75	2.03	60	1076	0.34	0	0.46	88	5	
6-02-75	0.51	15	74	0.34	0.56	0.10	22		
6-05-75	0.51	45	198	0.34	0	0.15	45		
6-05-75	0.30	5	113	0.34	0.56	0.15	19		
6-15-75	2.54	30	708	0.34	0	0.20	86		

*Avg. clarifier filling rate
(continued)

TABLE D-1. (continued)

Date	i (cm/hr)	t (min)	Q* (1/sec)	Gate Openings (m) Orifice	Overflow	Effluent	Water Level (%) Clar.	Tank	Comments
6-16-75				0.34					compressor
6-22-75	5.33	60		0.34	0	0	87	10	malfunction
6-26-75	0.51	15	269	0.34	0.56	0.46	14		
7-07-75	0.76	40	425	0.34	0.56	0.46	25		
7-10-75	1.83	10	425	0.34	0.56	0.46	20		
7-13-75	1.52	10	425	0.34	0.56	0.46	20		
7-19-75	1.52	10	241	0.34	0.56	0.10	48		
7-19-75	4.57	10	651	0.34	0.56	0.20	51		
7-20-75	3.05	10	396	0.34	0.56	0.20	46		
8-03-75	1.52	10	396	0.34	0	0.46	24		
8-03-75	2.67	20	765	0.34	0	0.46→0	86		
8-13-75	1.02	15		0.34	0	0	14		
8-13-75	4.57	5	538	0.34	0	0.26→0.13	21		
8-21-75	4.57	10	793	0.34	0.25	0.46	46		
8-24-75	3.05	15	680	0.34	0.56	0.46	43		
8-26-75	5.08	15	892	0.34	0.56	0.46	48		
8-29-75	0.57	40	71	0.34	0.56→0	0.46→0	30		
8-29-75	1.52	5	170	0.34	0.56	0.25	13		
8-30-75	3.05	5	368	0.34	0.56	0.25	33		
8-30-75	1.14	10	227	0.34	0.56	0.25	18		
8-30-75	1.52	15	453	0.34	0.56	0.25	44		
8-30-75	0.12	255		0.34	0	0	69		
8-31-75				0.34					rain data
8-31-75				0.34	0	0	70		lost
9-05-75	0.51	30	133	0.34	0	0	44		no notification
9-05-75	1.83	25	510	0.34	0.56→0	0.46→0	87		
9-05-75	3.43	20	708	0.34	0.56	0.46	49		
9-05-75	0.51	30		0.34	0.56	0.46	8		
9-11-75				0.34	0	0	20		test
9-11-75	1.78	30	334	0.34	0	0.10	55		

*Avg. clarifier filling rate
(continued)

TABLE D-1. (continued)

Date	i (cm/hr)	t (min)	Q* (l/sec)	Gate Openings (m) Orifice	 Overflow	 Effluent	Water Level (%) Clar.	 Tank	Comments
9-18-75	0.22	210	79	0.34	0.56→0	0.13→0	87	30	seepage test
9-20-75	1.02	15	227	0.34	0.56	0.25	24		
9-24-75	0.38	160		0.34	0.56	0.25	19		
10-08-75				0.34					
10-09-75				0.34		0	19		
10-15-75	1.22	15	297	0.34	0.56	0.10	48		
10-17-75	0.42	180	79	0.34	0.56→0	0.10→0	86	100	
10-17-75	0.20	150		0.34	0	0	86	100	
10-18-75	0.51	30		0.34	0.56	0.46	7		
11-05-75				0.34	0	0	22		DWF test
11-06-75				0.34	0	0	23		DWF test
11-07-75				0.34	0	0	23		DWF test
11-10-75	0.51	30	340	0.34	0	0.25→0	82		
11-29-75				0.34	0.56	0.10	38		rain data
11-30-75				0.34	0.56	0.10	25		lost
11-30-75	0.51	15		0.34	0.56	0.10	41		
11-30-75	0.76	10		0.34	0.56	0.10			
11-30-75				0.34	0	0	55		
11-30-75	2.29	10	337	0.34	0	0.10	49		
12-06-75				0.34	0.56	0.10	35		rain data
12-15-75				0.34	0.56	0.10	47		lost
12-15-75	0.46	5	91	0.34	0	0.10→0	84		
12-25-75	4.57	5	142	0.25	0.56	0.46	5		orifice at 10"
12-26-75	3.05	5	142	0.25	0.56	0.46	5		
12-30-75	3.05	15	170	0.25	0.56	0.46	6		
12-30-75	3.05	5	170	0.25	0.56	0.46	7		

*Avg. clarifier filling rate

TABLE D-2. SAMPLING DATA

Date	Location	Time	Total Solids (mg/1)	Suspended Solids (mg/1)	Settleable Solids (ml/1)	Volatile Solids (mg/1)	BOD (mg/1)	pH (S.U.)
4-01-74	Influent	12:30 AM						
		12:35 AM	17,500	600	7.5	40	110.	7.1
		12:40 AM	17,400	960	7.5	80	40.	7.1
		12:45 AM	18,100	320	6.0	40	190.	7.2
		12:50 AM	6,700	640	0.9	80	90.	7.2
		12:55 AM	5,500	320	0.5	40	90.	7.2
		1:00 AM	5,400	360	0.9	40	170.	7.0
		1:05 AM	5,500	320	0.3	40	80.	7.2
		1:10 AM	4,700	240	0.4	40	100.	7.2
		1:15 AM	5,000	320	0.3	80	100.	7.1
		1:20 AM	5,700	400	0.6	40	135.	7.2
		1:25 AM	4,900	280	0.5	80	150.	7.2
		1:30 AM	4,200	240	0.3	40	150.	7.1
		1:35 AM	4,300	320	0.4	80	110.	7.1
		1:40 AM	3,900	280	0.2	80	150.	7.0
		1:45 AM	3,900	200	0.2	40	100.	7.0
		1:50 AM	4,400	160	0.2	40	130.	7.1
		1:55 AM	4,200	720	0.2	40	70.	7.1
		2:00 AM	5,600	240	0.3	80	140.	7.0
		2:05 AM	4,900	240	0.3	80	90.	7.0
		2:10 AM	6,100	240	0.4	40	180.	7.2
		2:15 AM	14,800	240	0.2	80	130.	7.1
		2:20 AM						
		2:25 AM						
		Composite	9,400	800	1.5	80	190.	7.2
4-01-74	Weir	11:30 PM		1120	1.2	80	11.	7.2
		11:35 PM	Sampler Failed To Prime					
		11:40 PM	Sampler Failed To Prime					
		11:45 PM	Sampler Failed To Prime					

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
4-01-74	Weir	11:50 PM						
		11:55 PM	20,900	3,720	4.5	120	2.	7.3
		12:00 AM						
		12:05 AM	20,200	7,040	1.3	80	5.	7.4
		12:10 AM	24,300	9,800	1.6	160	1.	
		12:15 AM	19,500	3,000	2.5	N.D.	7.	7.5
		12:20 AM	16,600	2,000	2.5	200	8.	7.8
		12:25 AM	17,300	1,600	3.0	200	21.	7.4
		12:30 AM	15,500	1,000	2.8	100	9.	7.4
		12:35 AM						
		12:40 AM						
		12:45 AM	14,400	1,200	3.1	100	8.	7.4
		12:50 AM	20,600	1,300	1.5	200	7.	7.4
		12:55 AM	15,100	1,800	1.4	100	9.	7.4
		1:00 AM	12,200	1,400	1.2	N.D.	5.	7.4
		1:05 AM	14,700	1,200	1.9	N.D.	10.	7.4
		1:10 AM	13,200	1,300	2.0	N.D.	12.	7.4
		1:15 AM	12,300	1,000	2.1	100	8.	7.4
		1:20 AM	12,200	1,200	2.5	100	8.	7.4
		1:25 AM						
		Composite	22,100	2,300	3.5	300	5.	7.2
11-04-74	Influent	11:45 AM	416	312	2.5		15.4	7.2
		12:30 PM	232	168	2.2		16.8	7.0
		1:35 PM	224	120	2.0		15.8	7.1
		2:37 PM	459	164	0.5		4.0	7.4
		3:25 PM	436	76	0.3		5.2	7.1
		Composite	649	292	2.7		15.8	7.1

N.D.: not detectable

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
11-04-74	Weir	1:35 PM	962	363	5.5		17.2	7.1
		2:37 PM	421	178	1.3		17.0	7.1
		Composite	452	236	1.8		17.0	7.0
11-20-74	Influent	10:00 AM	147	116	0.5		16.8	7.2
		10:30 AM	298	122	0.3		16.5	7.1
		11:00 AM	194	22	0.4		11.7	7.2
		Composite	289	100	1.5		17.4	7.2
	Weir	10:00 AM	305	183	1.2		19.8	7.2
		10:30 AM	458	32	0.8		19.2	7.2
		11:00 AM	319	4	0.3		18.8	7.4
		Composite	431	114	1.2		17.1	7.2
1-08-75	Influent	4:45 AM	641	423	5.5	179	24.9	7.1
		4:50 AM	683	407	1.6	114	22.2	7.0
		4:55 AM	239	166	0.8	62	16.2	7.1
		5:00 AM	227	101	0.7	13	13.5	7.2
		5:05 AM	285	164	0.3	48	5.4	7.2
		5:10 AM	262	126	0.3	21	6.0	7.6
		5:15 AM	302	113	0.3	33	4.8	7.6
		5:20 AM	515	417	0.8	75	11.4	7.0
		5:25 AM	284	202	0.6	28	11.1	7.1
		5:30 AM	252	163	0.7	41	10.8	7.1
		5:35 AM	268	154	0.6	32	10.2	7.1
		Composite	394	298	0.9	63	15.6	7.1
	Weir	4:45 AM	416	312	2.0	66	22.8	7.1
		4:50 AM	346	214	1.0	74	23.7	7.1
		4:55 AM	309	200	0.9	48	22.2	7.2

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
1-08-75	Weir	5:00 AM	335	198	1.1	62	22.5	7.2
		5:05 AM	335	400	1.6	62	24.3	7.1
		5:10 AM	273	200	1.1	30	21.0	7.2
		5:15 AM	291	178	0.9	42	17.4	7.2
		5:20 AM	230	108	0.8	29	17.1	7.3
		5:25 AM	265	130	0.8	22	18.0	7.2
		5:30 AM	243	90	0.6	32	16.5	7.2
		5:35 AM	217	108	0.5	36	15.0	7.2
		Composite	290	140	0.8	46	17.7	7.1
1-08-75	Tank	9:00 AM	205	102	0.5	38	11.4	7.1
		9:05 AM	210	81	0.2	25	10.2	7.1
		9:10 AM	236	76	0.1	17	10.2	7.1
		9:15 AM	247	103	0.1	36	9.6	7.1
		9:20 AM	207	62	0.2	7	10.8	7.1
		9:25 AM	211	130	0.2	32	12.0	7.1
		9:30 AM	201	84	0.2	34	13.8	7.1
		9:35 AM	376	148	0.5	30	15.0	7.1
		Composite	217	72	0.3	12	11.4	7.1
2-23-75	Influent	5:45 PM	253	144	0.8	1	30.6	7.3
		5:55 PM	464	298	1.3	7	36.0	7.2
		6:05 PM	481	313	2.0	51	41.4	7.2
		6:15 PM	748	512	5.0	96	78.0	7.2
		6:25 PM	489	334	1.8	24	73.8	7.2
		6:35 PM	567	460	2.0	20	66.6	7.1
		6:45 PM	415	210	1.4	36	28.8	7.1
		6:55 PM	375	246	1.1	20	30.6	7.1
		7:05 PM	276	188	1.4	47	30.0	7.1
		7:15 PM	285	142	0.9	12	52.8	7.2

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
2-23-75	Influent	7:25 PM	227	132	1.0	44	30.0	7.2
		Composite	656	260	1.6	6	73.8	7.1
	Weir	5:45 PM	362	203	1.1	20	47.4	7.2
		5:55 PM	328	190	0.9	21	43.2	7.2
		6:05 PM	346	198	0.8	14	56.4	7.2
		6:15 PM	318	169	0.8	35	49.2	7.1
		6:25 PM	356	193	0.7	59	48.0	7.3
		6:35 PM	293	188	0.8	17	83.4	7.3
		6:45 PM	275	183	0.7	20	43.2	7.1
		6:55 PM	312	169	0.8	2	70.8	7.1
		7:05 PM	287	137	0.7	7	25.2	7.2
		7:15 PM	418	113	0.6	26	44.4	7.2
		7:25 PM	316	124	0.7	13	24.0	7.2
		Composite	371	201	1.0	8	48.0	7.2
2-23-75	Tank	8:45 PM	261	92	1.4	3	36.0	7.2
		9:00 PM	265	133	0.6	14	28.8	7.2
		9:05 PM	176	103	0.5	3	26.4	7.2
		9:10 PM	205	85	0.4	9	33.6	7.3
		9:15 PM	206	88	0.7	9	52.2	7.3
		9:20 PM	224	71	0.4	1	40.8	7.4
		9:30 PM	233	71	0.4	6	22.2	7.3
		9:35 PM	225	73	0.5	1	36.6	7.3
		9:45 PM	193	81	0.1	12	24.6	7.4
		Composite	246	129	0.3	10	25.2	7.2
5-20-75	Influent	10:50 AM	857	572	2.4	182	44.4	7.3
		10:55 AM	1253	1116	4.0	254	62.4	7.3
		11:00 AM	879	808	2.2	236	63.6	7.3

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
5-20-75	Influent	11:05 AM	575	458	1.5	108	62.4	7.0
5-21-75	Influent	5:25 PM	241	150	0.9	108	60.0	7.2
		5:30 PM	90	66	0.3	46	49.2	7.2
	Weir	5:35 PM	1326	1142	4.5	198	67.2	7.1
		5:40 PM	990	982	2.0	184	79.2	7.2
		5:45 PM	704	624	1.9	156	37.2	7.2
		5:50 PM	561	418	1.3	106	44.4	7.2
		Composite	1049	1044	3.5	240	48.0	7.2
5-31-75	Influent	4:40 PM	484	322	5.0	134		7.0
		4:45 PM	528	502	2.9	170		7.2
		4:50 PM	1313	1042	6.5	252		7.2
		4:55 PM	1148	1003	2.0	176		7.3
		5:00 PM	408	276	0.4	66		7.4
		5:05 PM	383	160	0.3	120		7.3
		5:10 PM	467	236	0.5	62		7.3
		5:15 PM	382	172	0.5	110		7.4
		5:20 PM	481	258	1.3	120		7.4
		Composite	761	562	2.5	122		7.2
	Weir	4:40 PM	1439	756	1.8	62		7.2
		4:45 PM	1229	640	1.1	60		7.3
		4:50 PM	1122	502	0.6	212		7.5
		4:55 PM	1038	382	0.6	90		7.7
		5:00 PM	988	456	0.7	110		7.7
		5:05 PM	1069	492	0.5	88		7.6
		5:10 PM	896	418	0.5	50		7.6
		5:15 PM	976	398	0.3	120		7.6

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
5-31-75	Weir	5:20 PM	767	330	0.6	138		7.5
		5:25 PM	794	292	0.3	114		7.5
		5:30 PM	632	154	0.2	30		7.5
		Composite	1851	898	1.5	148		7.4
5-31-75	Tank	5:30 PM	714	236	0.1	30		7.2
		5:35 PM	737	136	0.1	48		7.5
		5:40 PM	685	142	0.1	106		7.5
		5:45 PM	692	150	0.1	112		7.5
		5:50 PM	770	156	0.1	48		7.4
		5:55 PM	704	206	0.1	46		7.3
		6:00 PM	653	160	0.1	14		7.3
		6:05 PM	690	128	0.1	68		7.2
		6:10 PM	676	188	0.1	40		7.3
		6:15 PM	644	242	0.1	64		7.3
		6:20 PM	637	206	0.1	54		7.3
		Composite	632	130	0.1	48		7.5
6-22-75	Influent	7:00 PM	1035	720	3.2	184	13.2	7.0
		7:05 PM	1061	919	1.0	168	1.2	7.0
		7:15 PM	935	888	1.7	208	3.6	7.0
		7:20 PM	716	562	1.5	142	3.6	7.1
		7:25 PM	679	624	2.0	122	18.0	7.1
		7:30 PM	809	662	1.7	112	21.6	7.2
		Composite	532	524	1.6	84	20.4	7.1

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/1)	Suspended Solids (mg/1)	Settleable Solids (ml/1)	Volatile Solids (mg/1)	BOD (mg/1)	pH (S.U.)
6-22-75	Weir	6:35 PM	536	210	1.2	56	3.6	7.1
		6:40 PM	1071	834	2.3	92	24.6	7.2
		6:45 PM	2321	1668	3.4	156	24.0	7.2
		6:50 PM	810	644	1.5	104	13.2	7.2
	Tank	8:10 PM	403	280	1.2	86	7.2	7.2
		8:15 PM	205	126	0.9	48	4.8	7.2
		8:20 PM	264	80	0.6	66	12.6	7.1
		8:25 PM	292	88	0.7	40	14.4	7.2
	Overflow	7:55 PM	5037	4568	9.0	458	22.2	7.1
		8:00 PM	735	520	1.5	118	14.4	7.1
		Composite	467	468	1.2	142	12.6	7.0
9-05-75	Influent	2:50 PM	498	245	2.0		97.2	7.0
		2:55 PM	228	173	1.3		81.6	7.0
		3:00 PM	208	120	1.2		76.8	7.0
		3:05 PM	226	153	0.9		76.8	7.3
		3:10 PM	295	156	0.9		82.8	7.1
		3:15 PM	315	122	0.8		80.4	6.8
		Composite	294	179	0.9		79.2	7.0
	Weir	2:50 PM	471	96	1.5		63.6	7.1
		2:55 PM	631	302	1.5		60.0	7.0
9-18-75	Influent	9:10 AM	208	31	0.3		22.8	7.0
		9:15 AM	194	48	0.9		40.8	7.1
		9:30 AM	161	21	0.5		57.6	7.0
		9:35 AM	155	37	0.5		40.8	7.0
		10:45 AM	121	44	0.2		43.2	7.1

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
9-18-75	Weir	9:30 AM	179	19	0.3		32.4	7.3
		9:35 AM	162	13	0.1		48.0	7.1
		10:45 AM	214	20	0.1		70.8	7.0
	Tank	1:50 PM	149	79	0.2		47.4	7.1
		1:55 PM	191	39	0.3		39.6	7.1
		2:10 PM	166	20	0.2		30.0	7.1
10-17-75	Influent	4:20 PM	364	76	0.8		52.8	7.0
		4:25 PM	293	123	1.3		43.2	7.0
		4:30 PM	278	94	1.5		28.8	7.1
		4:35 PM	255	66	1.5		28.8	7.0
		5:00 PM	229	44	1.4		24.0	7.0
		5:30 PM	220	56	0.6		27.6	6.9
		6:00 PM	173	64	0.5		22.2	7.0
		8:40 PM	318	43	1.0		45.6	6.9
		8:55 PM	221	70	0.4		13.2	7.0
	Weir	4:25 PM	280	75	0.9		48.0	7.0
		4:30 PM	291	30	1.2		33.6	7.0
		4:35 PM	229	16	2.5		39.6	7.1
		4:40 PM	226	48	0.8		39.6	7.0
		5:00 PM	153	56	4.0		39.0	7.1
		5:30 PM	273	40	2.5		30.0	7.1
		6:00 PM	176	28	2.8		33.6	7.1

(continued)

TABLE D-2. (continued)

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
10-17-75	Tank	8:40 PM	305	75	0.2		28.8	7.1
		8:55 PM	145	39	0.1		27.6	7.1
		9:00 PM	127	27	0.1		33.6	7.1
		9:05 PM	89	19	0.1		25.8	7.0
		9:20 PM	194	10	0.1		19.2	7.0
		9:30 PM	211	39	0.2		28.8	7.0
		9:40 PM	283	16	0.2		30.0	7.0
		9:55 PM	312	12	0.8		46.8	6.9

TABLE D-3. SAMPLING DATA SUPPLEMENT

Date	Location	Time	Ammonia (mg/1)	Nitrate (mg/1)	Nitrite (mg/1)	Phosphorous (mg/1)	COD (mg/1)
4-01-74	Influent	12:30 PM					
		12:35 PM	N.D.	0.027	0.050	0.100	40
		12:40 PM	N.D.	0.356	0.048	0.275	
		12:45 PM	N.D.	0.190	0.067	0.315	
		12:50 PM	N.D.	0.072	0.038	0.068	
		12:55 PM	N.D.	0.165	0.052	0.125	N.D.
		1:00 AM	N.D.	0.185	0.057	0.120	
		1:05 AM	N.D.	0.145	0.016	0.305	
		1:10 AM	N.D.	0.165	0.028	0.170	
		1:15 AM	N.D.	0.146	0.012	0.115	
		1:20 AM	N.D.	0.375	N.D.	0.290	15
		1:25 AM	N.D.	0.025	0.032	0.290	
		1:30 AM	0.1	0.065	0.019	0.070	
		1:35 AM	N.D.	0.070	0.024	0.060	
		1:40 AM	N.D.	0.071	0.025	0.170	
		1:45 AM	N.D.	0.065	0.026	0.095	
		1:50 AM	N.D.	N.D.	0.028	0.500	
		1:55 AM	N.D.	N.D.	0.020	0.530	
		2:00 AM	N.D.	N.D.	0.029	0.240	
		2:05 AM	N.D.	N.D.	0.028	0.340	
		2:10 AM	N.D.	N.D.	0.038	0.240	
		2:15 AM	N.D.	N.D.	0.040	0.140	20
		2:20 AM	N.D.				
		2:25 AM					
		Composite	N.D.	N.D.	0.061	0.290	30
4-01-74	Weir	11:30 AM	Sampler Failed To Prime				
		11:35 AM	Sampler Failed To Prime				
		11:40 AM	Sampler Failed To Prime				
		11:45 AM	Sampler Failed To Prime				
		11:50 AM	Sampler Failed To Prime				

(continued)

TABLE D-3. (continued)

Date	Location	Time	Ammonia (mg/1)	Nitrate (mg/1)	Nitrite (mg/1)	Phosphorous (mg/1)	COD (mg/1)
4-01-74	Weir	11:55 PM	N.D.	0.010	0.020	0.150	35
		12:00 AM					
		12:05 AM	N.D.	0.010	0.025	0.21	
		12:10 AM	N.D.	N.D.	0.015	0.20	
		12:15 AM	N.D.	N.D.	0.035	0.20	25
		12:20 AM	N.D.	N.D.	0.025	0.14	
		12:25 AM	N.D.	N.D.	0.021	N.D.	N.D.
		12:30 AM	N.D.	0.025	0.020	0.07	
		12:35 AM					
		12:40 AM					
		12:45 AM	N.D.	0.005	0.023	N.D.	
		12:50 AM	0.01	0.015	0.016	N.D.	
		12:55 AM	N.D.	0.020	0.018	N.D.	
		1:00 AM	N.D.	0.016	0.017	0.41	
		1:05 AM	N.D.	0.005	0.022	N.D.	
		1:10 AM	N.D.	0.015	0.017	N.D.	
		1:15 AM	N.D.	0.015	0.020	N.D.	
		1:20 AM	N.D.	0.016	0.029	0.28	40
		1:25 AM	N.D.				
		Composite		N.D.	0.057	0.28	30
11-04-74	Influent	11:45 AM				0.001	
		12:30 PM				0.001	
		1:35 PM				0.013	
		2:37 PM				0.022	
		3:25 PM				N.D.	
		Composite	3.00	0.26	0.230	0.019	3.0
	Weir	1:35 PM				0.030	
		2:37 PM				0.015	
		Composite	0.50	0.44	0.34	0.010	10.0

(continued)

TABLE D-3. (continued)

Date	Location	Time	Ammonia (mg/l)	Nitrate (mg/l)	Nitrite (mg/l)	Phosphorous (mg/l)	COD (mg/l)
11-20-74	Influent	10:00 AM				0.100	
		10:30 AM				0.030	
		11:00 AM				0.020	
		Composite	1.29	.43	0.086	0.060	3.5
	Weir	10:00 AM				0.130	
		10:30 AM				0.125	
		11:00 AM				0.075	
		Composite	3.44	0.30	0.344	0.145	3.1
1-08-75	Influent	4:45 AM				0.31	
		4:50 AM				0.30	
		4:55 AM				0.22	
		5:00 AM				0.22	
		5:05 AM				0.33	
		5:10 AM				0.62	
		5:15 AM				0.48	
		5:20 AM				0.67	
		5:25 AM				0.45	
		5:30 AM				0.48	
		5:35 AM				0.55	
		Composite	1.01	0.66	0.023	0.43	320
	Weir	4:45 AM				0.25	
		4:50 AM				0.13	
		4:55 AM				0.16	
		5:00 AM				0.48	
		5:05 AM				0.43	
		5:10 AM				0.35	
		5:15 AM				0.62	
		5:20 AM				0.44	

(continued)

TABLE D-3. (continued)

Date	Location	Time	Ammonia (mg/1)	Nitrate (mg/1)	Nitrite (mg/1)	Phosphorous (mg/1)	COD (mg/1)
1-08-75	Weir	5:25 AM				1.58	
		5:30 AM				.34	
		5:35 AM				.26	
		Composite	1.31	.77	.052	.36	20
1-08-75	Tank	9:00 AM				0.54	
		9:05 AM				0.96	
		9:10 AM				0.39	
		9:15 AM				0.40	
		9:20 AM				0.35	
		9:25 AM				0.63	
		9:30 AM				0.37	
		9:35 AM				0.40	
		Composite	1.04	0.039	0.570	0.33	20
2-23-75	Influent	5:45 PM	1.95	0.33	0.156	0.22	
		5:55 PM	1.63	0.16	0.027	0.38	
		6:05 PM	0.57	0.06	0.033	0.32	
		6:15 PM	0.61	0.06	0.028	0.32	
		6:25 PM	0.86	0.07	0.028	0.50	
		6:35 PM	0.86	0.04	0.026	0.32	
		6:45 PM	0.59	0.04	0.033	0.35	
		6:55 PM	0.86	0.06	0.038	0.14	
		7:05 PM	0.93	0.05	0.042	0.19	
		7:15 PM	0.86	0.08	0.036	0.17	
		7:25 PM	1.00	0.07	0.043	0.30	
		Composite	0.75	0.05	0.046	0.19	80
	Weir	5:45 PM	1.25	0.18	0.037	0.04	
		5:55 PM	1.18	0.12	0.039	1.68	
		6:05 PM	1.35	0.12	0.038	0.07	

(continued)

TABLE D-3. (continued)

Date	Location	Time	Ammonia (mg/1)	Nitrate (mg/1)	Nitrite (mg/1)	Phosphorous (mg/1)	COD (mg/1)
2-23-75	Weir	6:15 PM	1.30	0.13	0.041	0.07	
		6:25 PM	1.25	0.11	0.036	0.06	
		6:35 PM	1.77	0.07	0.034	0.20	
		6:45 PM	1.60	0.10	0.029	0.18	
		6:55 PM	1.67	0.07	0.031	0.07	
		7:05 PM	1.60	0.12	0.034	0.04	
		7:15 PM	1.62	0.12	0.040	0.08	
		7:25 PM	1.51	0.12	0.037	0.05	
		Composite	1.60	0.11	0.041	0.16	130
2-23-75	Tank	8:45 PM	1.0	0.315	0.028	0.23	
		9:00 PM	1.25	0.255	0.032	0.05	
		9:05 PM	1.00	0.200	0.100	0.64	
		9:10 PM	1.30	0.185	0.029	0.08	
		9:15 PM	0.90	0.700	0.041	0.01	
		9:20 PM	0.64	0.265	0.025	0.01	
		9:30 PM	0.80	0.170	0.025	0.16	
		9:35 PM	0.78	0.445	0.032	0.11	
		9:45 PM	1.06	0.540	0.035	N.D.	
		Composite	0.80	0.305	0.025	0.07	160
5-20-75	Influent	10:50 AM	1.0			0.25	
		10:55 AM	3.6			0.56	
		11:00 AM	2.9			0.61	
		11:05 AM	2.8			1.04	
5-21-75	Influent	5:25 PM	2.5			0.30	
		5:30 PM	2.5			0.42	
5-21-75	Weir	5:35 PM	3.6			0.76	
		5:40 PM	2.7			0.48	

(continued)

TABLE D-3. (continued)

Date	Location	Time	Ammonia (mg/l)	Nitrate (mg/l)	Nitrite (mg/l)	Phosphorous (mg/l)	COD (mg/l)
5-21-75	Weir	5:45 PM	1.6			0.31	
		5:50 PM	1.4			0.27	
		Composite	2.7			0.62	
6-22-75	Influent	7:00 PM	0.50	0.015			
		7:05 PM	0.97	0.017			
		7:15 PM	0.43	0.017			
		7:20 PM	0.67	0.023			
		7:25 PM	0.85	0.025			
		7:30 PM	1.55	0.052			
		Composite	0.56	0.035		4.10	
	Weir	6:35 PM	0.72	0.030			
		6:40 PM	1.23	0.050			
		6:45 PM	1.10	0.049			
		6:50 PM	1.00	0.032		0.81	
	Tank	8:10 PM	0.60	0.036			
		8:15 PM	0.50	0.029			
		8:20 PM	0.55	0.032			
		8:25 PM	0.65	0.024			
	Overflow	7:55 PM	0.73	0.062			
		8:00 PM	0.40	0.014			
		Composite	0.27	0.015		1.45	

TABLE D-4. GRAB SAMPLING DATA

Date	Location	Time	Total Solids (mg/l)	Suspended Solids (mg/l)	Settleable Solids (ml/l)	Volatile Solids (mg/l)	BOD (mg/l)	pH (S.U.)
11-15-73	Clarifier	5:00 PM	840	640	10.0	40	840	6.9
12-13-73	Overflow	9:25 AM	1640	1290	0.8	30	350	
12-13-73	Clarifier	9:25 AM	1520	1300	0.7	10	320	6.8
12-13-73	Influent	9:25 AM	1067	1040	2.0	20	260	
1-23-74	Clarifier	11:00 AM	340	120	0.2	40	75	6.8
3-22-74	Clarifier	9:00 AM	65	20	7.5	TRACE	101	7.6
4-02-74	Overflow	2:30 AM	2500	200		120	97	7.4
5-31-74	Clarifier	1:00 PM	593	84	4.0	4	18	7.2
9-03-74	Clarifier	10:00 AM	308	70	2.5	6	14	7.3
1-08-75	Clarifier	9:00 AM	295	158	0.4	14	11	7.1
2-23-75	Clarifier	8:30 PM	307	100	1.5	3	58	7.2
2-23-75	Overflow	8:30 PM	276	115	0.4	33	46	7.2
5-20-75	Clarifier	10:55 AM	543	246	0.7	176	65	7.1
5-20-75	Weir	10:55 AM	440	174	2.0	148	70	7.2
5-31-75	Clarifier	5:30 PM	476	116	0.1	30		7.6
5-31-75	Weir	5:53 PM	676	168	0.1	48		7.5
6-22-75	Overflow	8:15 PM	467	212	1.5	136	15	7.0

TABLE D-5. GRAB SAMPLING DATA SUPPLEMENT

Date	Location	Time	Ammonia (mg/1)	Nitrate (mg/1)	Nitrite (mg/1)	Phosphorous (mg/1)	COD (mg/1)
11-15-73	Clarifier	5:00 PM	3	1	N.D.	0.22	750
1-23-74	Clarifier	11:00 AM	1	0.62	0.62	0.08	180
3-22-74	Clarifier	9:00 AM	10	4.05	0.28	2.45	210
4-02-74	Overflow	2:30 AM	N.D.	N.D.	0.04	0.35	50
5-31-74	Clarifier	1:00 PM	6.00	0.27	0.42	N.D.	57
9-03-74	Clarifier	10:00 AM	8.50	1.25	0.12	0.25	20
1-08-75	Clarifier	9:00 AM	0.98	0.03	0.78	0.62	60
2-23-75	Clarifier	8:30 PM	1.00	0.19	0.04	0.24	
2-23-75	Overflow	8:30 PM	0.63	0.19	0.03	0.05	
5-20-75	Clarifier	10:55 AM	4.45			1.20	
5-20-75	Weir	10:55 AM	4.35			2.05	
6-22-75	Overflow	8:15 PM	0.25	0.02		1.13	